
01 Jan 1971

Column research council: proceedings 1971

Column Research Council

Follow this and additional works at: <https://scholarsmine.mst.edu/ccfss-library>



Part of the [Structural Engineering Commons](#)

Recommended Citation

Column Research Council, "Column research council: proceedings 1971" (1971). *Center for Cold-Formed Steel Structures Library*. 218.
<https://scholarsmine.mst.edu/ccfss-library/218>

This Technical Report is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in Center for Cold-Formed Steel Structures Library by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

CCFSS LIBRARY Fritz Engineering Foundation
19 5 * 4906 COLUMN RESEARCH COUNCIL
1971 PROCEEDINGS

CCFSS LIBRARY Fritz Engineering Foundation
19 5 * 4906 COLUMN RESEARCH COUNCIL
1971 PROCEEDINGS

=	



COLUMN RESEARCH COUNCIL

Established in 1944
by the Engineering Foundation

Proceedings 1971

The 1971 Annual Meeting and Technical Session were supported
by a grant from the National Science Foundation

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania 18015

Table of Contents

FORWARD	1
THE CRC EXECUTIVE COMMITTEE 1971	2
ANNUAL TECHNICAL SESSION	3
Abstracts of Oral Presentations	5
Panel Discussion27
Annual Business Meeting56
Participation in Technical Meetings59
Revision of the CRC Guide60
Appendix63
LIST OF PUBLICATIONS69
FINANCE72
Cash Statement and Budget72
REGISTER74
Officers74
Executive Committee74
Standing and Ad Hoc Committees74
Task Groups75
Task Reporters80
Participating Organizations81
Representatives of Participating Organizations83
Members-at-Large85
Corresponding Members86
Addresses of Participants87
BY-LAWS AND RULES OF PROCEDURE	103

Foreword

The Column Research Council has as its purpose to study and discuss problems related to the stability of metal compression elements and metal structures. This involves the stimulation and organization of research, the dissemination of information on stability problems and, last but by no means least, the formulation of design criteria for use by designers and specification writers. The Column Research Council consists of dedicated and friendly people who have a common interest in being a bridge between the knowledge gained by researchers and the knowledge needed by designers.

The activities of the Council involve Task Group activities, committee meetings and an Annual Meeting. These Proceedings tell of these activities, they list the people and organizations who take part in them, and it is a record of the Annual Meeting which took place in Pittsburgh in May 1971.

The technical contributions and the financial support given by individuals and organizations have made the continued vitality of the Column Research Council possible, and their efforts are very much appreciated.

While many individuals or groups could be mentioned here, this will not be done. Their names and accomplishments are in the record of these Proceedings. I only want to single out Dr. Bruce Johnston, who has throughout this year, tirelessly worked on the Third Edition of the "Guide". This effort is singularly worthwhile, and I gratefully acknowledge Dr. Johnston's contribution to it.

The sponsorship and financial support of the National Science Foundation for the Annual Technical Session is gratefully acknowledged.



T. V. Galambos, Chairman
Column Research Council

The CRC Executive Committee

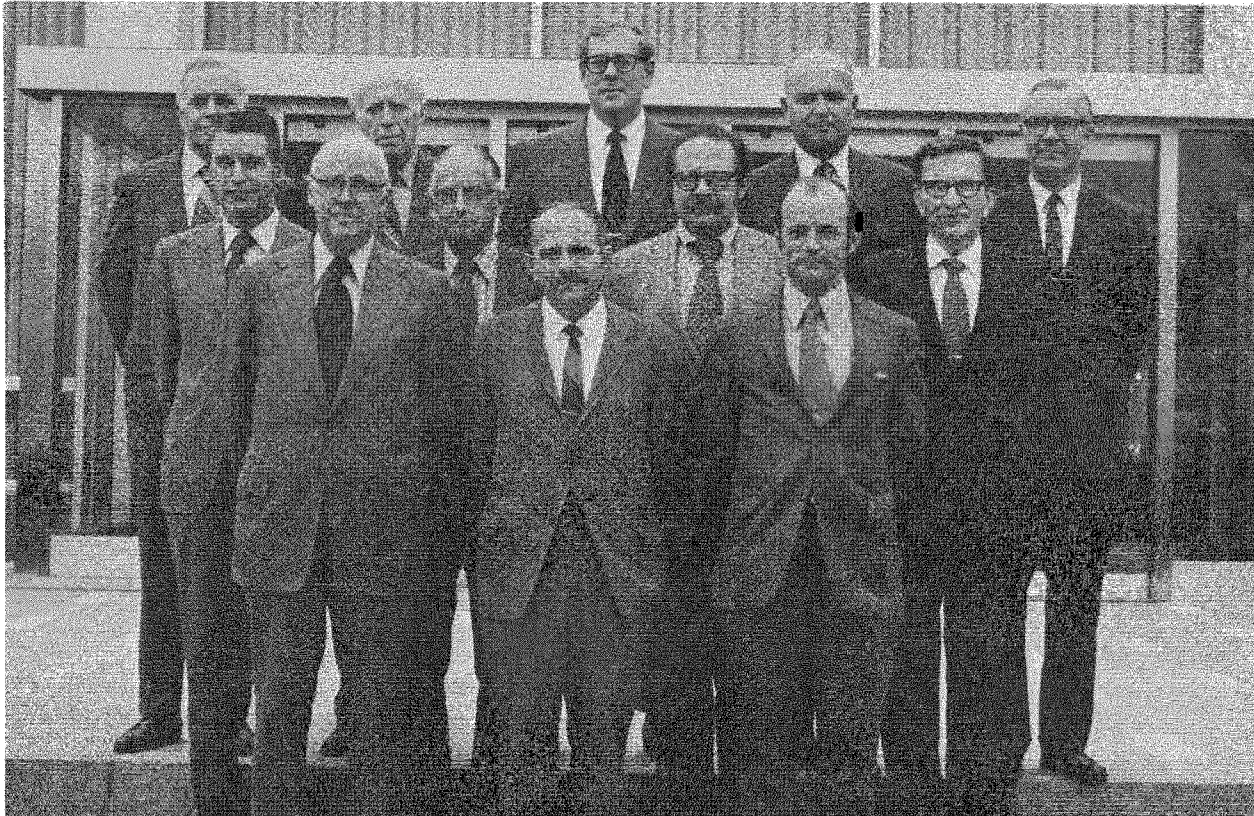
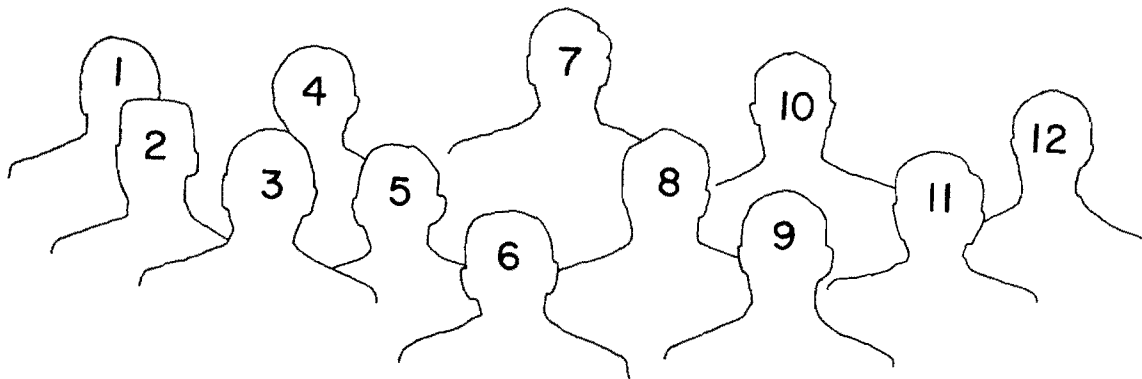


Photo taken September 30, 1971 at the National Bureau of Standards, Washington, D. C.



- | | | |
|--------------------|----------------------|----------------------|
| 1 - E. H. Gaylord | 5 - B. G. Johnston | 9 - F. Van der Woude |
| 2 - L. K. Irwin | 6 - T. V. Galambos | 10 - W. A. Milek |
| 3 - J. A. Gilligan | 7 - J. S. B. Iffland | 11 - J. W. Clark |
| 4 - T. R. Higgins | 8 - I. M. Hooper | 12 - L. S. Beedle |

Members not in the photo: T. Dembie, J. L. Durkee, C. F. Scheffey, G. Winter

Annual Technical Session

One of the purposes of the Council is to maintain a forum where problems related to the design and behavior of columns and other compression elements in metal structures can be presented for evaluation and discussion. The Annual Technical Session provides opportunity to carry out this function.

The 1971 Annual Technical Session was held on May 25 and 26 at the Pick-Rossevelt Hotel in Pittsburgh, Pennsylvania. Seventy-seven persons attended the Session and twenty-two papers were delivered.

A panel discussion on "Fire Effects on Structural Stability" was held in the evening of May 25.

In conjunction with the Technical Session, an Annual Business Meeting was held for the purpose of electing new officers and members, and to discuss financial and other business matters.

Abstracts of the technical papers, a transcript of the panel discussion, and minutes of the business meeting are recorded in the following pages. The attendance list is also included.

T A S K G R O U P R E P O R T S

TASK GROUP 1, CENTRALLY LOADED COLUMNS

Chairman, J. A. Gilligan, United States Steel Corporation

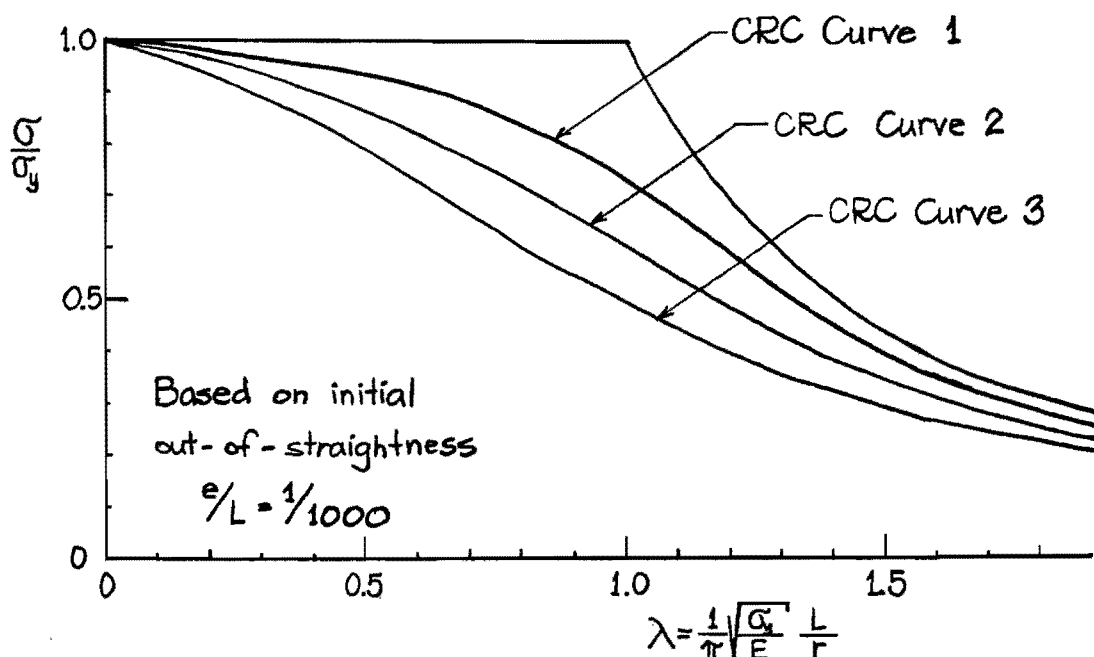
Maximum Column Strength and the Multiple Column Curve Concept

R. Bjørhovde and L. Tall, Lehigh University

The results of a study on the variation of the maximum column strength is presented, taking into account factors such as residual stress, yield strength, cross-sectional properties, and out-of-straightness. The data were obtained using a computer program based on an iterative, incremental procedure, wherein any distribution of the residual stress and the yield strength can be accepted.

Column curves representing rolled and welded wide-flange shapes and welded box-shapes in seven different steel grades, in addition to some hybrid sections and some annealed sections, have been developed, giving a total number of 102 curves. Statistical analyses of the band of curves have been performed throughout the range of slenderness ratios, indicating a fairly good agreement with available test results.

Based on the results of this study, three column curves have been developed, using an initial out-of-straightness equal to $L/1000$. The three curves have been recommended to CRC Task Group 1 for the adoption as the new CRC column strength curves. Pending the approval by the CRC, the three curves will replace the currently used CRC Column Strength Curve, which is based on the tangent modulus load. The recommended curves are shown in the figure below.



The Maximum Column Strength Curves

Task Group Reports

TASK GROUP 4, FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

Chairman, J. S. B. Iffland, Praeger-Kavanagh-Waterbury

Report of Activities of Task Group 4

J. S. B. Iffland

Task Group 4 is involved in the following two areas of endeavor:

1. Preparation of Chapter 15, Frame Stability, for the 3rd Edition of the CRC Guide.
2. Stimulating and guiding research in the effective length and frame stability areas.

Three drafts of Chapter 15 have been prepared by Task Group 4 and this chapter is nearly in condition to submit to the Guide Editor. A task group meeting was held on May 23, 1971.

Research is actively being engaged in at:

1. New York University
2. Lehigh University
3. Cooper Union
4. University of Wisconsin
5. University of Alberta

A coordination meeting was held between participants of the first three schools in New York on May 11, 1971. Reports on the work at Lehigh University, Cooper Union and the University of Alberta are subjects of separate reports at this technical session.

The Sway Increment Method of Frame Analysis

J. H. Daniels, Lehigh University

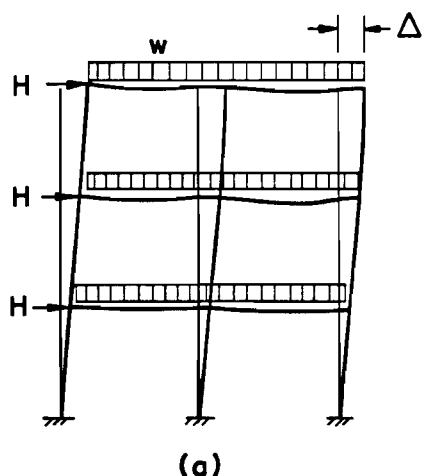
An exact analytical procedure is presented for determining the complete elastic-plastic behavior of unbraced multi-story steel frames which are subjected to nonproportional combined loading. The procedure is called the sway increment method of analysis and is based on determining the values of the applied lateral loads consistent with prescribed finite sway deflections of a story when the frame is also subjected to constant gravity loads. The analytical method utilizes a second-order elastic-plastic method of analysis, an incremental procedure, and a technique to predict the sway increments for next hinges. The procedure includes the effects of axial shortening, hinge reversal and residual stresses.

Task Group Reports

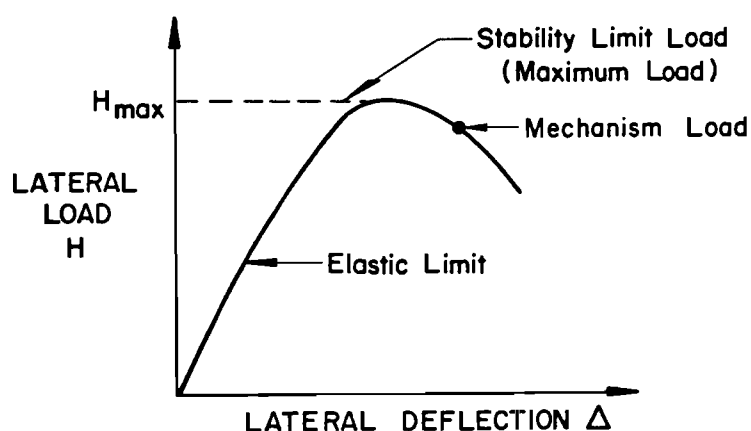
The sway increment method is used to study the lateral-load versus sway-deflection behavior of several multi-story frames under nonproportional combined loading. These studies indicate that the effect of axial shortening on the maximum lateral load capacity is not considerable and the primary effect of axial shortening is to induce lateral deflections. Plastic hinge reversals hardly occur in a frame before reaching its maximum lateral load unless there are any plastic hinges in the non-swayed position with the gravity loads only. A number of plastic hinges in a frame is subjected to hinge reversals after failure. However, the effect of the hinge reversals on the unloading behavior of the frame is very small.

Based on the sway increment method, an approximate method, which is called the one-story assemblage method, is developed to determine the approximate lateral-load versus sway-deflection behavior of a story of an unbraced frame. This method which is programmed for computer solution is very useful for performing the trial analyses associated with preliminary frame designs. The individual story behavior obtained using the one-story assemblage method has been compared with the story behavior determined from a sway increment analysis for several stories in two frames studied. The comparison indicates that the one-story assemblage method gives a reasonably good approximation to the load-deflection behavior of a story located in the middle and lower regions of an unbraced frame subjected to nonproportional combined loads.

The results of both the sway increment method of analysis and the one-story assemblage method of analysis are compared with experimental results. The agreement between the experimental results and the theoretical predictions is good.



(a)



(b)

Typical Load-Deflection Behavior of an Unbraced Frame under Nonproportional Combined Loading.

Task Group Reports

Elastic Buckling Analysis of Space Frames

Shosuke Morino, Lehigh University

First a determinantal approach for obtaining the critical load of a space frame is introduced. This approach makes use of the concept that the determinant of the overall stiffness matrix of the frame goes to zero as the applied load reaches the critical value. A bound of the critical load can be established for some cases in which the value of the determinant approaches zero and changes its sign as the applied load increases. It is, however, shown that for frames with more than one axis of symmetry the determinant may not change its sign even if the applied load exceeds a certain critical load. A different bounding technique based on examining the eigen-values of the overall stiffness matrix is discussed.

Also presented is the effect of warping on the elastic buckling strength of space frames. Several sample frames are solved according to three types of deteriorated twisting stiffness, each for a given type of boundary condition for warping.

Stability Design of Steel Frames Under Combined Loads

Le-Wu Lu, Lehigh University

Work is being carried out at Fritz Engineering Laboratory, Lehigh University to develop a frame design method that will take into account in a direct manner the effect of frame instability. The $P-\Delta$ moment which causes the frame instability effect is closely examined in two practical building frames. Some preliminary results indicating the interrelation between the working load drift limitation and the lateral load carrying capacity of the frames are also included.

Stability Studies of Braced Frames

J. H. Davison, University of West Virginia and P. F. Adams, University of Alberta

This study considers the behavior of tall building frames subjected to combined vertical and horizontal loads or to vertical loads alone. The frames may be unbraced or may be braced by shear walls or by a diagonal or K type bracing systems. The frames are analyzed by using a second order elastic-plastic analysis which is able to consider the influence of the axial loads as well as the finite joint size. The results are presented in the form of load displacement diagrams.

Task Group Reports

TASK GROUP 7, TAPERED MEMBERS

Chairman, A. Amirikian, U. S. Naval Facilities Engineering Command

Design Recommendations for Tapered Structural Members

George C. Lee, State University of New York at Buffalo

The presentation summarizes the results of analytical and experimental studies on tapered structural members at Buffalo and the development of design specifications that are recommended by Task Group 7 (jointly with the tapered member subcommittee of the Welding Research Council). The proposed allowable stress formulas are applicable to the proportioning of members with linearly tapered webs only. No ultimate strength design was considered.

The basic approach used in the development of design formulas was as follows: firstly, theoretical solutions are obtained, then - using these solutions - the A.I.S.C. prismatic member design formulas are modified by the introduction of appropriate multiplying factors dependent only on the tapering geometry to effect the same solutions. These factors reduce to unity when there is no taper in the member. This approach assumes that the current A.I.S.C. allowable stress formulas for prismatic members are adequate.

For a detailed description of this study, the following reference may be consulted:

Lee, G. C., Morrel, M. L., and Ketter, R. L. "Design of Tapered Members", Welding Research Council Bulletin (in press).

TASK GROUP 8, DYNAMIC INSTABILITY

Chairman, D. A. daDeppo, University of Arizona

Comparative Studies of Unified Finite Element Techniques for Dynamic Instability Analysis of Frameworks

F. Y. Cheng, University of Missouri - Rolla

Three general methods classified as frequency dependent stiffness, consistent mass, and discrete mass are formulated for investigating the effect of conservative axial forces on dynamic response and dynamic characteristic values of structural systems. The time-dependent lateral forces may be concentrated, uniform, or non-uniform, and are formulated in load matrices based on beam-column interaction behavior. General

Task Group Reports

considerations in each of these three methods include the rotatory inertia, shear and bending deformations, and the second-order of axial loads.

The objective of this report is to show the upper and lower bounds of solutions obtained by using the methods presented. A general recommendation is made for correct choice of the methods.

TASK GROUP 10, DESIGN OF Laterally UNSUPPORTED BEAM-COLUMNS

Chairman, T. V. Galambos, Washington University

The Post-Buckling Behavior of Laterally Unsupported Beam-Columns

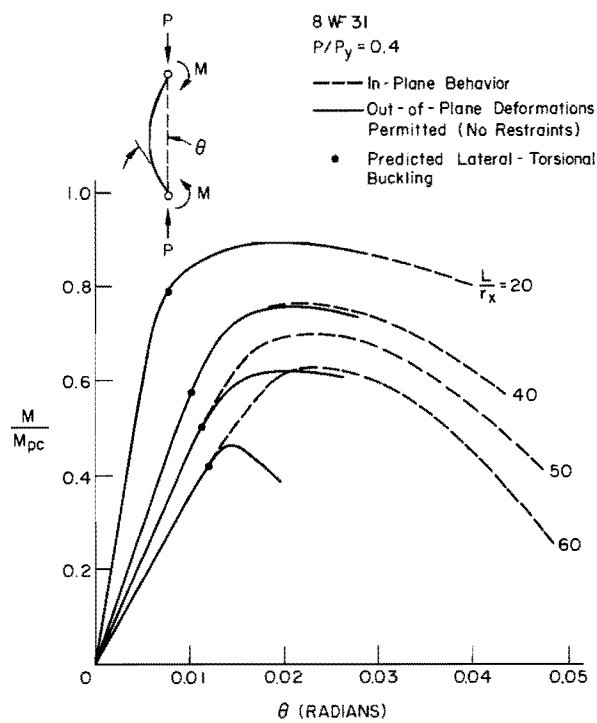
L. C. Lim, LeMessurier Associates

The behavior of laterally unsupported as-rolled WF beam-columns after buckling has been investigated theoretically and experimentally. The concept of initial imperfections is used to obtain the moment-rotation relationship of beam-column after the occurrence of lateral-torsional buckling. The theoretical solutions show that short columns have substantial post-buckling strength and rotation (see Fig. 1). Long columns tend to unload soon after buckling. The analytical results are compared with the available experimental results and good correlations are obtained. A comparative study shows that the current CRC interaction formula for laterally unsupported columns is conservative for columns with small slenderness ratio. The following design formula is proposed:

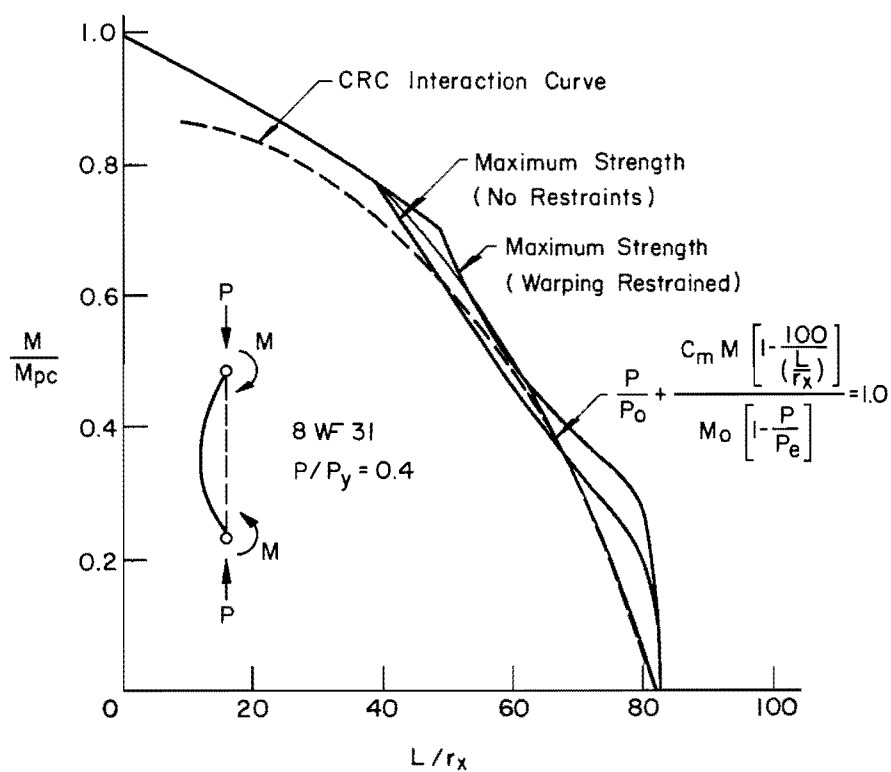
$$\frac{M}{M_m} \leq \frac{M_o \left[1 - \frac{P}{P_e} \right] \left[1 - \frac{P}{P_o} \right]}{C_m M_m \left[1 - \frac{100}{\left(\frac{L}{r_x} \right)^2} \right]} \quad \text{or} \quad \leq 1.0, \text{ the smaller of the two.}$$

In the above equation, M_m is the in-plane moment capacity of the beam-column. The terms M_o , P_o , P_e , and C_m are as defined in the CRC Guide.

Task Group Reports



Moment-Rotation Relationships for
Laterally Unbraced 8W31 Beam-Columns



Comparison of Interaction Curves for
8W31 Beam-Columns

Task Group Reports

TASK GROUP 11, EUROPEAN COLUMN STUDIES

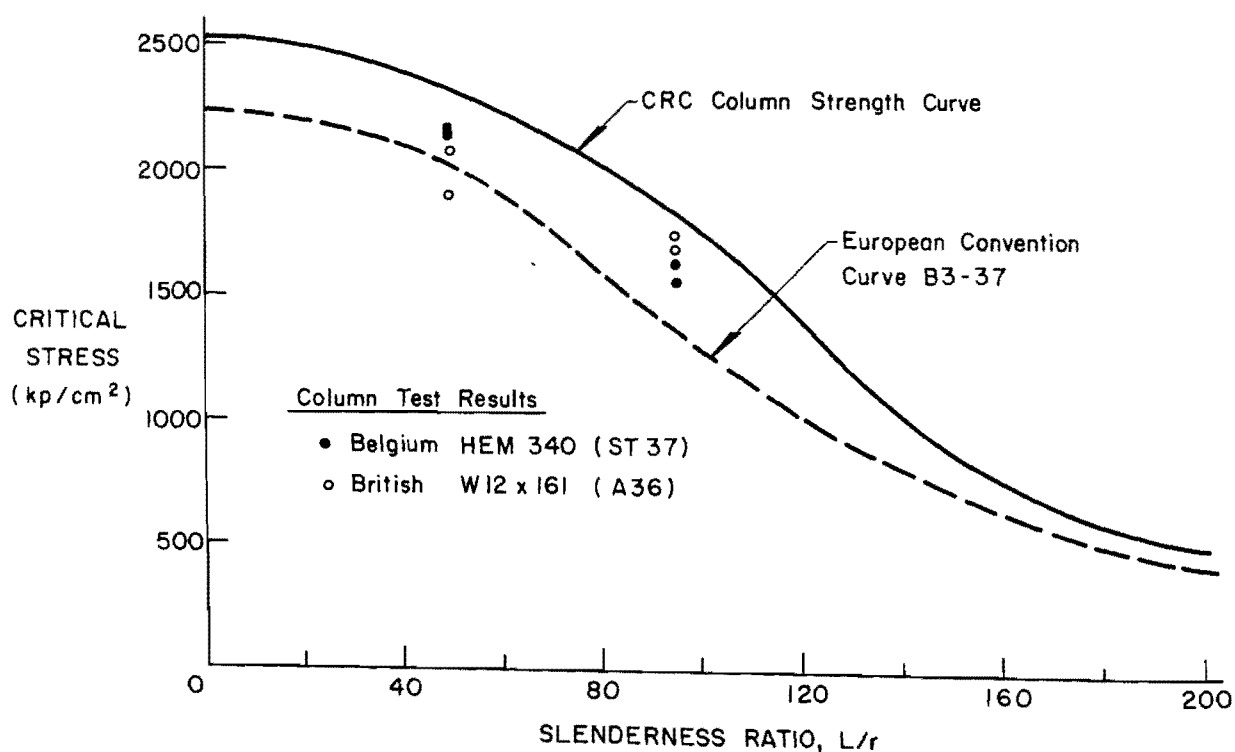
Chairman, D. Sfintesco, CTICM, France

European Column Tests - Progress Report

N. Tebedge, Lehigh University

In order to obtain conclusive experimental evidence on the strength of heavy columns with minimum cost, the program is restricted to testing specimens from four countries: Belgium, Britain, Germany and Italy. The test program consists of column tests (slenderness ratio of 50 and 95) and supplementary tests, namely, tension tests (full-size and ASTM standard), residual stress measurement, and stub column test.

Tests on the specimens from Belgium and Britain have been completed, and the specimens from Germany and Italy are being tested. The test results are compared with the latest proposed European Convention curve for the particular shape and the CRC column strength curve as shown below.



Comparison of Column Test Results (HEM 340) with Proposed ECCS Curve and CRC Column Strength Curve.

Task Group Reports

TASK GROUP 13, THIN-WALLED METAL CONSTRUCTION

Chairman, S. J. Errera, Bethlehem Steel Corporation

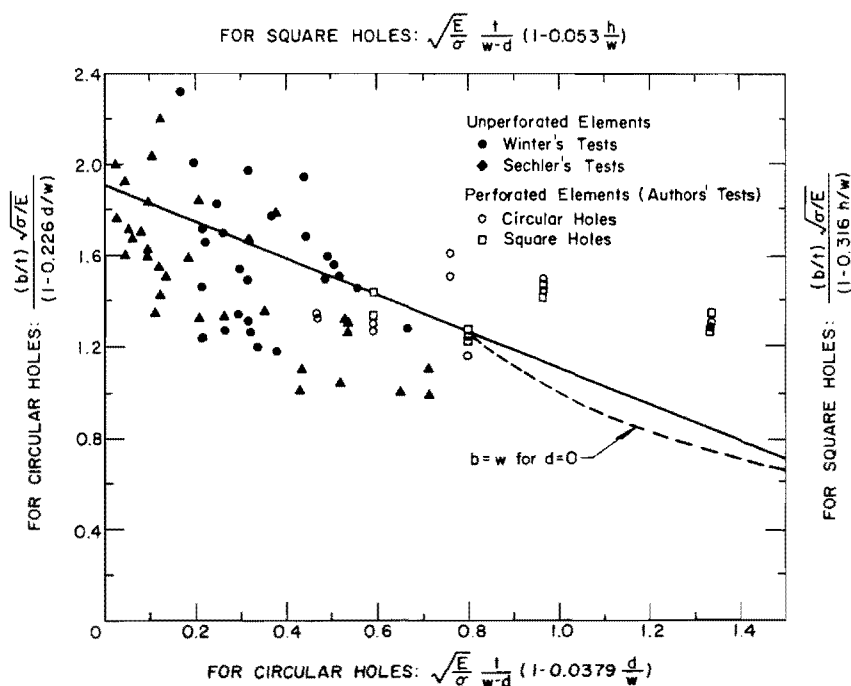
Structural Stability of Cold-Formed Steel Compression Members Having Perforated Stiffened Elements

W. W. Yu and C. S. Davis, University of Missouri - Rolla

This presentation describes the study of buckling behavior and post-buckling strength of cold-formed steel columns having circular and square perforations in stiffened elements. Analytical results were verified by the test data obtained from the experimental investigation.

It was found that Winter's effective width equation can be modified for use in determining the effective width of perforated stiffened compression elements.

Even though the buckling load for the stiffened elements is affected more by the square holes than circular holes, the post-buckling strength of the elements with square and circular perforations were found to be nearly the same.



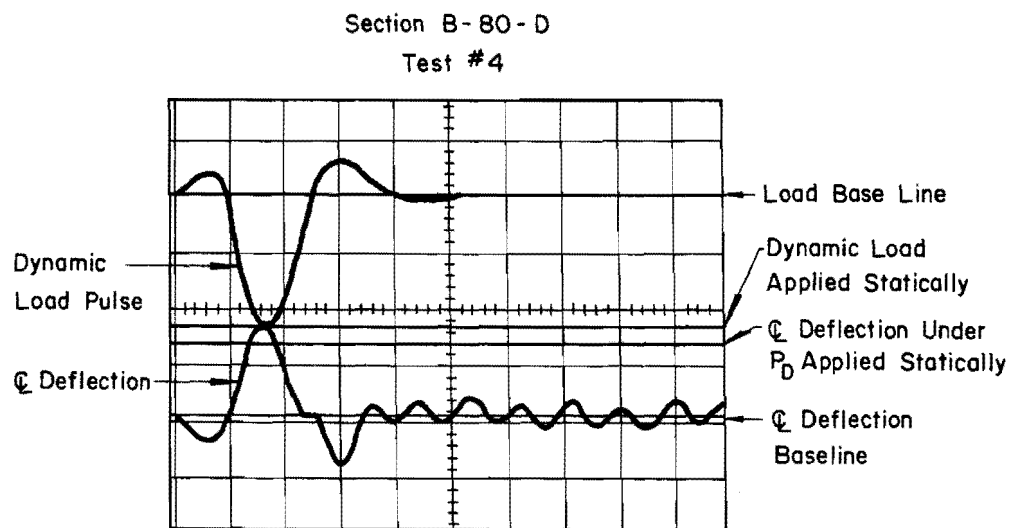
Comparison of Effective Width

Task Group Reports

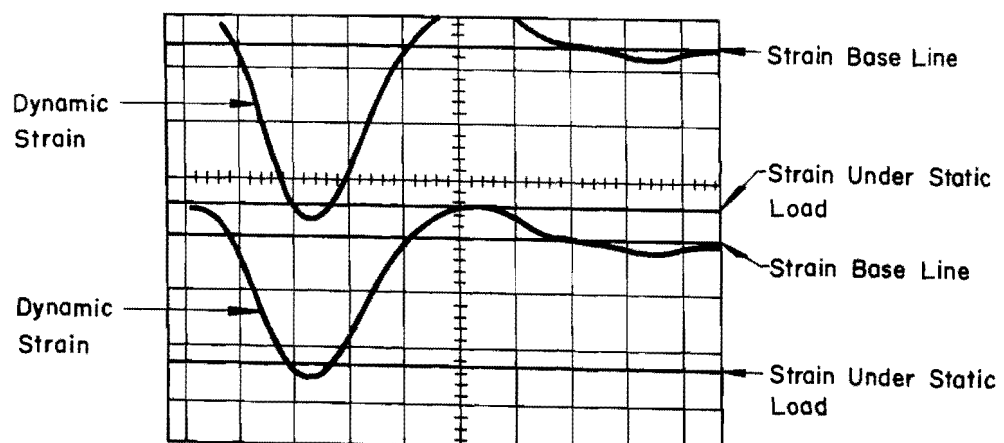
Impact Loading of Thin-Walled Cold-Formed Columns

C. Culver, Carnegie-Mellon University

Test results for static and dynamic loading of thin-walled cold-formed columns are presented. Columns subjected to combined local and overall buckling ($Q < 1$) as well as columns subjected only to overall buckling ($Q = 1$) were tested. The static ultimate loads are compared with existing design requirements for these columns. The experimental behavior of the columns subjected to short duration impact loads is described and compared with the behavior under static loading.



(a) Load, Deflections



(b) Strains

Task Group Reports

TASK GROUP 16, BUILT-UP GIRDERS

Chairman, F. D. Sears, Department of Transportation

Testing of Rectangular Model Box Girders

J. A. Corrado and B. T. Yen

The objectives of the experimental work were to observe the failure modes of steel box girders and to obtain some stress magnitudes in such girders for an analytical analysis. The models were 2 ft. long with 3 x 4 in. cross section and a wider top flange plate. Transverse stiffeners were used on the webs. One model was subjected to both concentric (symmetrical) and eccentric (unsymmetrical) loads. It was observed that tension field action of plate girder web panels took place in the two web plates, either simultaneously or sequentially depending on the loading condition. Failure of the box girder occurred after both the webs and the flanges failed. Testing of a second model was being conducted at the time of this report.



Model Box Girder After Testing

Task Group Reports

IABSE Colloquium on Design of Plate and Box Girders for Ultimate Strength

J. W. Clark, Aluminum Company of America

The IABSE Colloquium in London on March 25 and 26 brought together 20 research workers from nine countries to discuss ultimate strength design of plate and box girders. Technical papers by the participants were distributed before the meeting and will be published in the Proceedings of the IABSE. The countries represented were Belgium, Czechoslovakia, France, Great Britain, Japan, Sweden, Switzerland, United States and West Germany. Contributions were also received from the USSR and Poland, although their representatives did not attend the colloquium.

Some of the topics discussed were shear strength of unstiffened girders (stiffeners at supports only); resistance of unstiffened girders to concentrated loads on the flange; strength of transversely stiffened girders under shear, bending, and combined bending and shear; effect of longitudinal stiffeners on girder strength; design of unsymmetrical girders; lateral buckling strength; fatigue; and ultimate strength of stiffened box girders. In some cases, different investigators had approached the same problem in different ways - for example, the effect of flange stiffness on girder strength - and there was lively discussion on the relative merits of the different approaches.

While the colloquium perhaps did not bring about complete agreement on the best methods of handling the various problems, it provided all the participants with a much better insight and understanding of the significant contributions being made in many countries to the solution of these important structural problems.

Major Strength Theories for Plate Girders

A. Ostapenko, Lehigh University

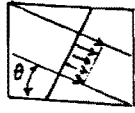
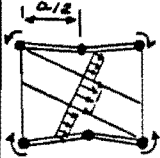
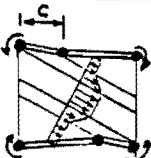
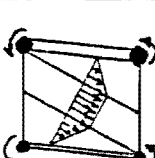
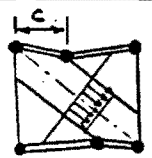
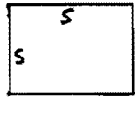
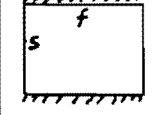
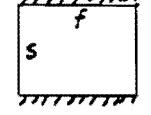
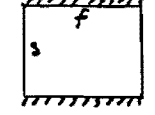
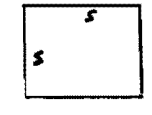
Considerable research has been conducted on the ultimate strength of plate girders since Basler and Thürlimann offered their first formulation in the early sixties. Of particular interest was the formulation of an analytical model which would improve on their model. The IABSE London Colloquium "Design of Plate and Box Girders for Ultimate Strength" served as a forum for the latest theories (March 1971). The basic analytical models and main assumptions of those theories are briefly summarised in the table for the case of pure shear.

The assumed pattern of the tension field stresses and the deformation of the flanges are shown by sketches on Line 1. As indicated on Line 2, the web plate is assumed to be either simply supported at all edges (Cols. 1 and 6) or simply supported at the stiffeners and fixed at the flanges (Cols. 2 to 4). Lines 3 to 6 give comments on some other assumptions pertaining to the shear strength, and Lines 7 and 8 on the extensibility of a particular model to the case of

Task Group Reports

combined action of shear and bending or to longitudinally stiffened plate girders.

It is noteworthy that in spite of sometimes contradictory assumptions made by the individual authors, many experiments confirm these theories quite well.

		BASLER	FUJII	KOMATSU	CHERN & OSTAPENKO	ROCKEY & SKALLOUD
		1	2	3	4	5
1	Shear Strength Mechanism					
2	Edge Conditions for Web Buckling					
3	Tension Field Band Width is Function of:	θ	θ	θ	θ	Distance c. It is a function of Z_f , plast. str. of flanges
4	Intensity of Outer TFB depends on:	Zero	Z_f	c (from tests) and Z_f	TFB Stress	Zero
5	Contribution of Flange Strength, Z_f , through:	Not Considered	Intensity of Outer TF Band	Intensity and Variation of OTF Band	Frame Action Shear	Width of TF Band
6	Unequal Flanges (Unsymm. Gdr.)	No Effect	Not Applicable	Not Applicable	Considered	Not Applicable
7	Analytical Extension to M & V (combined loads)	No	No	No	Yes Buckl. of C. Fl. or Yldg. of Web to Neutral Axis	Yes Web Yldg. at Flange
8	Inclusion of Long. Stiff.	Yes (Cooper)	Yes, for shear	Yes, shear. Mid-depth.	Yes M & V	A suggest. made.

Theoretical Models Proposed for Analysis of Ultimate Strength of Plate Girders

Task Group Reports

TASK GROUP 17, STABILITY OF SHELL-LIKE STRUCTURES

Chairman, K. P. Buchert, University of Missouri at Columbia

Research Needs in Shell-Like Structures

K. P. Buchert, University of Missouri at Columbia

Shell-like structures are being used more and more for roof structures, cooling towers, nuclear power plants, underwater structures and other similar applications. Unfortunately, the designer has little information available to him to assist in the stability analysis and design. The purpose of this paper is to outline some of the problems that need attention. Perhaps the greatest need is in the area of shell-like structures with negative Gaussian curvature (The Gaussian curvature is the product of the two curvatures of the shell surface). The greatest need is for solutions to the hyperbolic paraboloid and the hyperboloid of revolution.

For shells of positive curvature solutions are needed for the conoid, freeformed shell, translational shell and paraboloid of revolution.

Some solutions are available for zero curvature shell-like structures. However, many of the solutions are available for aerospace type structures and often these are not applicable to those structures used by civil and mechanical engineers.

Unfortunately, the computer solutions that have been proposed to date (1971), are often of little value in the analysis of the stability of shell-like civil engineering structures.

In order for a solution to be of much value to civil engineers it must at least consider the following items: 1. plasticity reduction factor, 2. deflections, 3. fabrication and erection tolerances and 4. edge conditions. In addition, such items as restricted wave length of buckle, joint details, local buckling and member buckling should be considered.

The Column Research Council could make a valuable contribution by considering these items in detail and by promoting more research in shell-like structures.

Dynamic Plasticity of Clamped Circular Plates

D. Krajcinovic, Argonne National Laboratory

In numerous applications of technical significance a structural element, such as a clamped circular plate, is subjected to the action of

Task Group Reports

blast loading. It is of interest, therefore, to analyze the response of such a plate, and in particular to determine the ensuing plastic deformation being a certain indicator of the plate strength.

This paper considers a rigid perfectly plastic circular plate clamped along the entire perimeter. The pressure is uniformly distributed across the entire plate surface. Using power series in the very beginning of the deformation process a suitable computational scheme is established. Results are computed for various pulse shapes. Next, the effective load and the mean time (being the first moment of the time load function) are chosen as correlation parameters.

$$P_e = \frac{I}{2t_{\text{mean}}}$$

where

$$I = \int_{t_y}^{t_f} p(t) dt \quad t_{\text{mean}} = \frac{1}{I} \int_{t_y}^{t_f} (t - t_y) p(t) dt$$

with t_y , t_f being the times when the plastic deformation begins and terminates respectively, and $p(t)$ being the function describing the pressure time history. Finally, it is shown that, using the correlation parameters, the influence of pulse shape is for all practical purposes eliminated. Therefore, knowing deformation for, say, rectangular pulse shape, the analyst can simply, using formulas listed above, compute deformation for an arbitrary pulse shape.

Applications of Reticulated Hyperbolic Shells

D. R. Sherman, University of Wisconsin at Milwaukee

In the design of reticulated shell-like structures, three types of potential instability failures must be considered:

1. buckling of individual members
2. local buckling or snap through of a joint
3. general buckling involving several joints

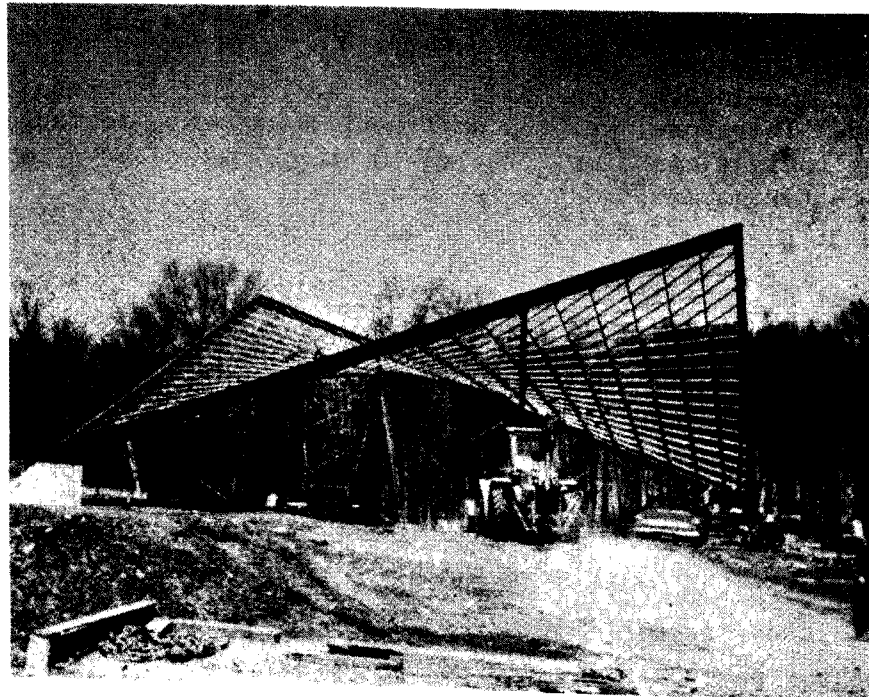
These problems have been formulated for grids, single curvature and dome structures, and design oriented solutions are available which are in fairly good agreement. However, few solutions can be found for local and general instability problems of hyperbolic structures, even though several of these structures have been built.

Reticulated hyperbolic structures have been built in the form of single sheet hyperboloids and hyperbolic parabolas. A few single sheet hyperboloid roof and tower structures were built in the early 1960's,

Task Group Reports

primarily outside the U. S. Some of the larger towers are over 300 feet high.

The first reticulated hypar roofs also were built in the early 60's. These early roofs were small test structures to demonstrate the feasibility and check the force distribution. Since then, many small roofs with spans less than 50 feet have been built for architectural effect. Intermediate spans of about 100 feet have been used for auditorium type structures and recently large hangar facilities with 200 foot spans have been constructed. One of the newest developments is the stressed skin hypar module which is being used in large hangar facilities and has been proposed as an efficient form for many other applications. In much of this construction, tests have been used to verify the capacity of the structure. Proven theories regarding the design of reticulated hyperbolic structures to resist instability failure are needed.



Reticulated Hypar Roof

Task Group Reports

TASK GROUP 18, TUBULAR MEMBERS

Chairman, A. L. Johnson, American Iron and Steel Institute

Design Criteria for Structural Steel Pipe

P. W. Marshall, Shell Oil Company, New Orleans

Structural steel pipe is used in the construction of fixed offshore platforms and similar tubular structures. While small members may utilize manufactured tubes, larger members (over 3/8 in. wall and 16 in. diameter) are fabricated from plate steel by cold forming and welding. Residual stresses and imperfections are introduced, which affect the behavior of tubular beam-columns, and influence the selection of design criteria.

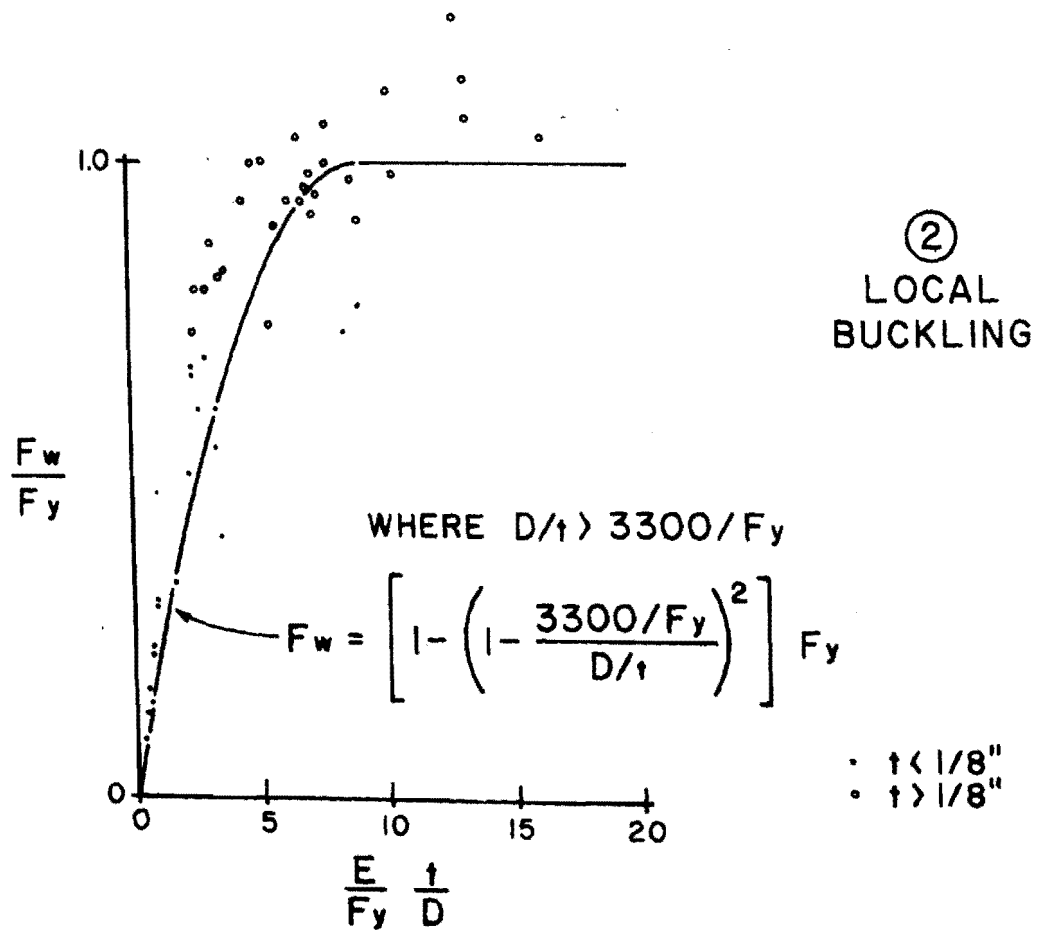
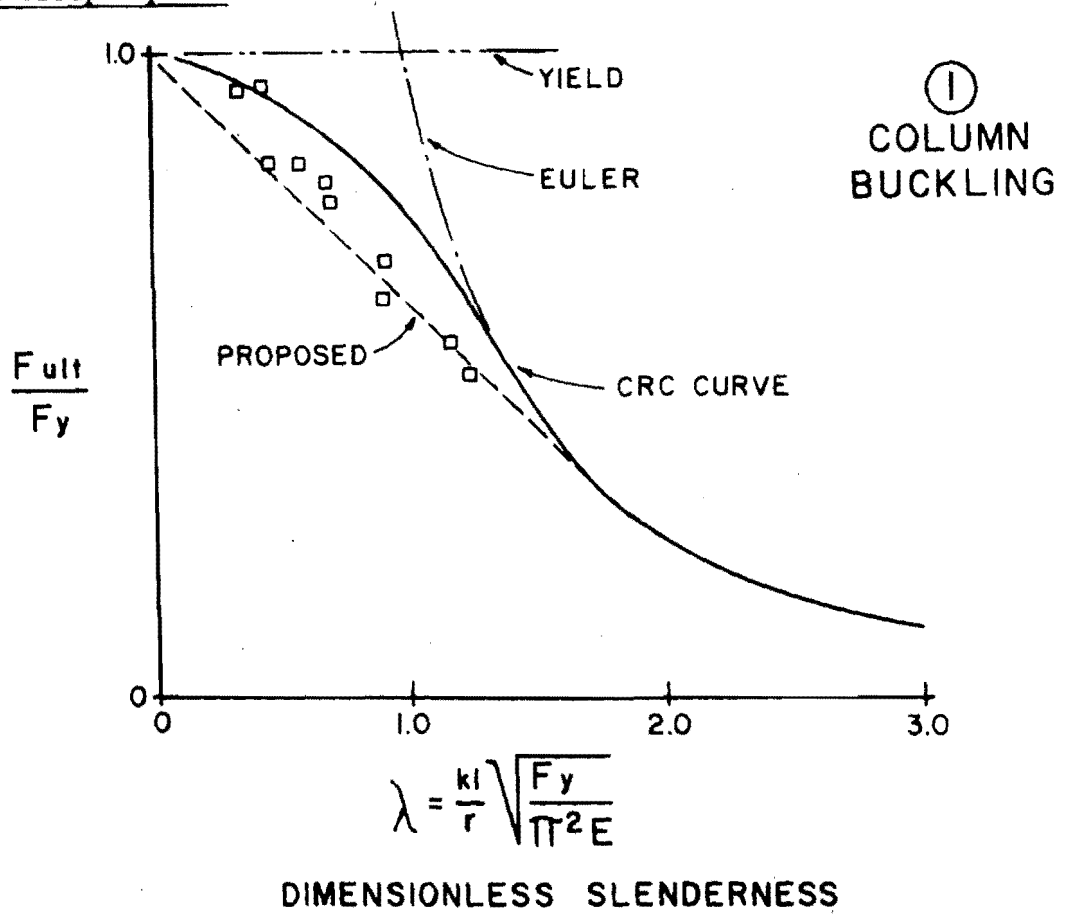
Column buckling may be considered in terms of normalized failure stress vs. dimensionless slenderness (Figure 1). Data for welded square tubes indicates that a design curve of the type proposed by Schilling (1964) may be more appropriate for tubular columns than the CRC curve (which is incorporated in the AISC code used by many designers).

Local buckling considerations are indicated in Figure 2. This is a plot of the local wrinkling stress F_w (normalized on yield) versus a dimensionless thickness parameter, which is derived from classical local buckling theory and permits rational common treatment of various yield strengths. Test data (1846-1946) and a proposed design curve are shown in the figure.

For tubular compression members in which there is interaction between column buckling and local buckling, the approach taken by section C5 of the AISC code, which is tantamount to substituting F_w for F_y in the appropriate column formulae, appears to be most reasonable.

In flexure, circular tubes appear to belong to a class of "semi-compact" members. That is, most practical sections can develop the fully plastic moment, but may lack sufficient rotation capacity to justify ultimate strength design. Progressive failure - ovalization of the cross section in the region of the initial plastic hinges, prior to complete redistribution of bending moments - was predicted in the case of a fixed ended beam loaded at the third points, using empirical moment-curvature data with an elasto-plastic finite element computer program.

A number of useful research programs dealing experimentally with the behavior of fabricated tubular columns and flexural members are suggested by the foregoing.

Task Group Reports

CONTRIBUTIONS OF TASK REPORTERS

TASK REPORTER 11, STABILITY OF ALUMINUM STRUCTURAL MEMBERS

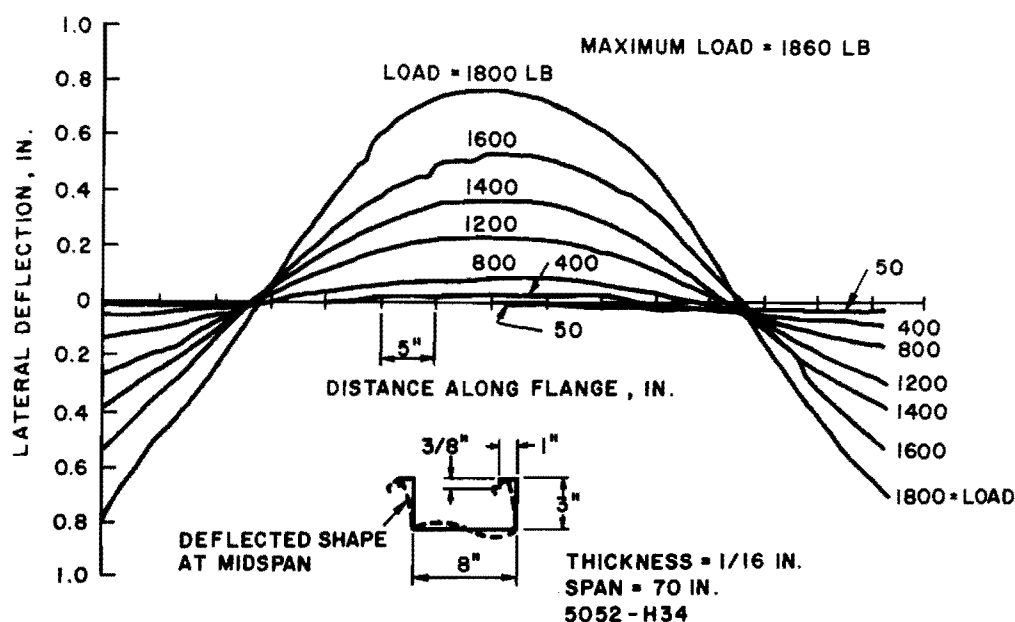
J. W. Clark, Aluminum Company of America

Aluminum Members With Elastically Restrained Compression Flanges

M. L. Sharp, Alcoa Research Laboratories

Aluminum structures such as curtain walls, cable trays, sign panels, and ship hulls often incorporate stiffening members which have compression flanges that are not supported against lateral buckling. The purpose of this experimental and analytical investigation is to establish means of analysis of members of this type.

Formed sheet specimens of alloy 5052-H34 were tested in bending. Twelve cross-sectional configurations and two thicknesses of material, 1/8 in. and 1/16 in., were included. In all cases the measured lateral deflections of the compression flanges increased with load and thus no abrupt buckling behavior occurred. A representative record of results is given in the figure. The lateral deflection was due in part to the eccentricity of loading caused by the fact that the flanges were unsymmetrical. Comparisons of ultimate strengths obtained from tests with the method of analysis as given in the Light Gage Cold-Formed Steel Design Manual - AISI and with a torsional buckling analysis showed that these methods tended to overestimate the test strengths. The overestimation of strength for the aluminum members apparently occurred because the effects of the lateral bending of the flanges and the large distortions of the cross-sectional shape are not considered in these buckling analyses.



LATERAL DEFLECTION CURVES

TASK REPORTER 13, LOCAL INELASTIC BUCKLING

L. W. Lu, Lehigh University

A literature survey was made in the local buckling of steel members in the elastic and inelastic range. The survey includes wide-flange sections, circular tubes, and rectangular tubes used as beams, columns and beam-columns. It is found that solutions are still needed for several cases of elastic buckling (for example, circular tube subject to uniform or non-uniform bending). Little information is available on the buckling of a partially yielded web element in a wide-flange beam-column. Solutions are also needed for inelastic buckling of circular and rectangular tubes under bending or combined bending and axial thrust.

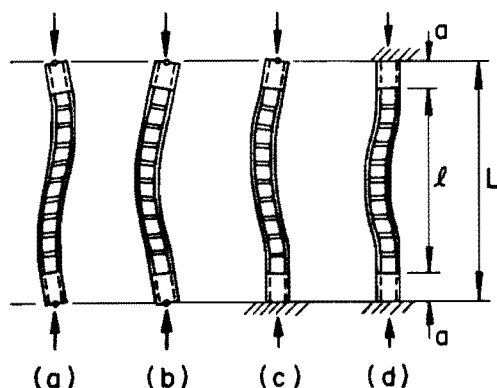
RESEARCH REPORTS

Spaced Steel Columns

B. G. Johnston, University of Arizona

End tie plates in battened columns may contribute significantly to the buckling strength. Their effect is accentuated by the study of a spaced column, defined herein as the limiting case of a battened column in which the battens are attached to the longitudinal column elements by hinged connections. The battens then act simply as spacers, with no shear transmitted between the longitudinal elements. Without end tie plates, the buckling strength of such a spaced column is no greater than the sum of the critical loads of the individual longitudinal components of the built up member. The strengthening effect of the end tie plates is due to two factors: (1) A shortening of the length within which the column components can bend about their own axes and (2) the longitudinal components are forced to buckle in a modification of second mode shape and thus have elastic buckling coefficients that approach four times those of the first mode. The buckling load of a spaced column with end tie plates is a lower bound to the buckling load of a battened column with low or uncertain moment resistance in the connections between battens and the longitudinal components.

For the hinged end condition the spaced column with end tie plates will buckle either in Mode A (center reversal of curvature) [Fig. (a)] or in Mode B (semi-fixed shape) [Fig. (b)], depending on the values of I/I_0 and a/L . It will be noted that in Mode A buckling there is no differential change of length between the end tie plates; thus the two longitudinal column components may buckle in Mode A under identical loads $P/2$, and the critical load is independent of the ratio I/I_0 . When the column buckles in Mode B [Fig. (b)], the shortening under column load is greater on the concave side than on the convex; thus there is an added internal resisting moment due to direct forces in the components that is added to the bending moments induced in the components themselves. The critical loads for Mode B buckling may be less than those for double curvature when the ratio I/I_0 is relatively small and a/L is large.



Spaced Column Buckling Modes: (a) Hinged-Hinged, Mode A, (b) Hinged-Hinged, Mode B, (c) Hinged-Fixed, (d) Fixed-Fixed.

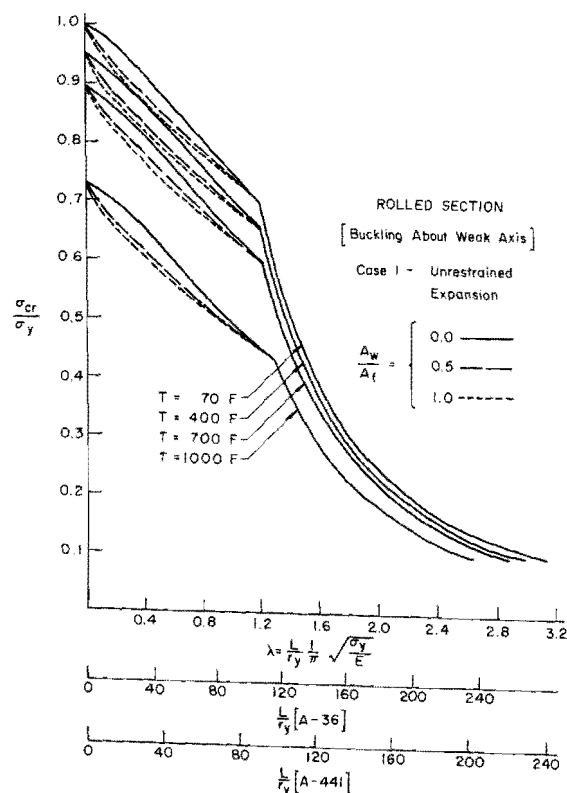
Research Reports continued

In practice, the base of a column will usually be attached to a footing, and buckling in Mode A cannot take place. Buckling will be in the shape as shown in Fig. (c) but as I/I_0 gets large it will tend toward the shape with both ends fixed, as shown in Fig. (d). In fixed end buckling [Fig. (d)], as in hinged end, the resisting moment is simply the sum of the moments in the component parts, with no contribution due to differential direct forces as in Figs. (b) or (c). The fixed end case is the simplest to evaluate; the critical load is simply twice the critical load of a longitudinal component, of length L , with both ends fixed; i.e., the Euler load with an equivalent length of $0.5L$, multiplied by two.

Column Buckling at Elevated Temperature

C. Culver, Carnegie-Mellon University

The buckling strength of rolled steel wide flange columns subjected to fire temperatures is discussed. The decrease in strength due to the reduction of elastic modulus and yield strength associated with elevated temperature is discussed. Column curves and simple formulas relating column strength at elevated temperature to that at room temperature are presented. The results are compared with present design requirements for allowable column temperatures under fire loading.



Buckling Strength of Wide Flange Columns
at Elevated Temperature

P A N E L D I S C U S S I O N

FIRE EFFECTS ON STRUCTURAL STABILITY

Panel Members:

L. G. Seigel, United States Steel Corporation
G. V. Smith, Consulting Engineer
A. F. Nassetta, Weiskopf and Pickworth
R. Bletzacher, Ohio State University

MR. TILMANS

Good evening, gentlemen. My name is Anthony Tilmans. I am the chairman of the Structural Division of the Pittsburgh Section of ASCE and in conjunction with the Column Research Council we are co-sponsoring this evening's panel discussion on fire effects on structural stability. We are very happy and proud to be able to co-sponsor this program. ASCE is always happy to sponsor conferences with such distinguished speakers and topics and it gives me great pleasure to be involved in situations of this type.

Without further ado, I now present Mr. John Gilligan of the United States Steel Corporation who will introduce the panel members and continue with the panel discussion.

MR. GILLIGAN

I want to thank the Pittsburgh Section of ASCE for co-sponsoring this panel. We have four panelists who will make 10 to 15 minute presentations.

This is not a new problem. Fire has been around for a long time. What's new about it is that we have new techniques and new information available which will enable a large segment of this audience to start making a worthwhile examination of the effects of fires on all sorts of structures. We are not going to limit ourselves to buildings -- any structure that might experience a fire would come under this discussion. Some of the impetus for this panel discussion and for the interest in the subject, I believe, has been stimulated by the development of new fire-protection systems. I feel that what is needed is good input on fire loading, the properties of materials, how some of the practical problems are being handled, and what can and should be done in areas of structural engineering and research. These are the titles that each of our panelists will speak on.

Panel Discussion

Mr. Gilligan, Cont'd

After being with the panelists earlier this evening I realized we have a serious communication gap. I recall, at a meeting with Column Research Council not too long ago, somebody suggested that we should investigate fire loads. I'm sure that certain people in the group thought this was a new static loading condition that occurred during a fire.

All but one of our panelists came prepared to talk solely on columns. So we all have a lot of catching up to do. Now let us try to put it all together and put it to good use.

Following the presentations we will have time for discussion and I hope you will enter into this freely.

To start this off, speaking on fire loads, fire protection and fire tests is Larry Seigel. Larry received his Bachelor of Mechanical Engineering at the University of Pittsburgh; Master of Science, Mechanical Engineering, Case Institute of Technology. He's had quite a varied background of experience; 4 years with the Cleveland Electric Illuminating Company; 13 years Associate Professor of Heat Power Engineering, Case Institute; 5 years as Head of the Engineering Department, Gannon College at Erie, and concurrent with this he was consultant to the Bureau of Ships for 18 years. He was in private consulting practice for 4 years and in 1964 he joined the U. S. Steel Corporation to work on problems of fire protection of buildings. Larry, please give us the benefit of your knowledge on fire loads, protection, and tests.

MR. SEIGEL

I am happy to have this opportunity to discuss the problem of temperature effects on building structures with you this evening.

Temperature effects are important because they may cause thermal stresses or movements in buildings, or in other structures, that require special consideration by structural engineers. Temperature changes may occur due to ordinary circumstances, or they may be the result of an accident such as a fire. Tonight I intend to spend most of the time discussing accidental conditions, particularly fires, but I did not want to neglect mentioning that other conditions do occur and that many structural engineers are already familiar with how to handle them.

One of the most common situations of this type is the case where exterior members in a building that are subjected to the ambient conditions that vary in temperature from winter to summer while interior members are maintained at essentially a constant temperature year round. Obviously, some stresses occur or some movements occur, or probably

Panel Discussion

Mr. Seigel, Cont'd

combinations of each, and these things can be troublesome, particularly in very large or very tall buildings. This is a typical problem that is frequently encountered and solved by good structural engineering. However, this is not the type of problem I intend to discuss this evening.

Instead, I would like to discuss the effects of severe localized temperature differences that may occur in buildings, or any structure, due to fires. These effects may occur in refinery areas where there are various pieces of process equipment. Or they may occur in buildings. My own experience has been particularly related to fires in buildings and so the examples that I give will be particularly related to buildings, but the principles involved are by no means related specifically to buildings.

The effect of high temperature exposure of a building to fire is seldom dealt with by the structural engineers, particularly because the building codes are so specific in how to deal with the matter. If one were to develop an excellent but unusual design of a fire resistant building, it is questionable whether the building officials would be willing to accept it. In fact, I should put it another way: whether they could accept it. Because to the building official the building code is a legal document. It does many things for him. But most importantly, it provides him with a set of laws to enforce. A good building code, I have been informed, is one which makes it possible for a building official to make decisions without having to do anything other than rely on standard tests that are prescribed within the building code. So effectively a building official checks to see that all standards are met, and he seldom makes original decisions. So if there is a lot of similarity in building design today, one reason for it is that building codes demand it.

In one respect, the situation with regard to the building codes is a happy state of affairs in that it places no demands on the structural engineer. However, closing one's eyes to a problem does not accomplish its solution, nor does it make the problem vanish. Recent experience in fires in high-rise buildings suggests that our traditional code and standard approach may not be working as well as it ought to. So something better is certainly in order for discussion and that is the purpose of tonight's session.

To define the problem, I will give a brief summary of the present test criteria for fire resistant construction, and then to consider how closely these criteria may suit the conditions during fire exposure in an actual building. The fire resistance of a structural member is defined by building codes in terms of the time that a member can withstand exposure to a standard fire without collapse of the member. That sounds simple enough. And it sounds as though it is almost sensible, and because it sounds so good it has been in existence for about 80 years in this

Panel Discussion

Mr. Seigel, Cont'd

country. During this period there have been no significant changes. But it is time to consider changes.

Fires in buildings do not burn according to a standard, and full size building members do not perform like test specimens. In the United States the standard fire exposure is defined by ASTM Standard E119. It is a time-temperature relation that is maintained within a furnace, and the member to be tested is simply placed in the furnace for a period of time that is required to meet the code. If certain criteria are met, a fire resistance rating is established and the design is accepted for use in building construction. Now of course real fires may differ significantly from test fires, but the standard test fire provides means for comparing different materials and protection systems. That is the main purpose of the standard test. And it is not just an American custom. There are similar time-temperature relations and test procedures in existence all over the world and they are often remarkably alike.

To be a little more specific, for a steel building column, the standard test specimen in the United States is a 10 in. wide flange 49 lb. column 9 ft. long. It is put into a test chamber. A standard fire, which is furnished by gas, is applied to the chamber, and the column may be loaded to its design load. It is tested until it fails structurally. The fire resistance rating achieved is given in even hours or half-hours, so that if failure occurred at 130 minutes the column would be assigned a two hour rating; if failure occurred at 119 minutes the rating would also be a two hour rating; but if failure occurred at 118 minutes the rating would be one and one-half hours. So minutes are very important in fire testing.

But not all columns are tested under load. An alternate method which uses a temperature limit of 1,000°F average steel temperature, with a maximum of 1,200°F has also been established. By comparing the performance of measured test results under load with the time of failure when a 1,000°F average temperature is reached testing laboratories have demonstrated that there is reasonable agreement of both methods.

Of course, there are different kinds of furnaces used for testing the various members: the wall furnace for walls, the column furnace for columns, and a floor furnace for floor-ceiling assemblies. All of these have their size limitations and these limitations should be of considerable interest to structural engineers. Because certain limitations have to be placed on how large a test assembly can be built, there is often a problem of justifying the performance of the actual construction in a building compared to its performance in the test. There is evidence that the performance is not always the same. The effects that are important from a structural standpoint are those that result from increased temperature of the steel members and from the interaction

Panel Discussion

Mr. Seigel, Cont'd

of members with each other in a building as opposed to an assumed performance that may be maintained until a temperature of 1,000°F is reached. Also, even if members are tested under load, their performance may be different when subjected to a fire in an actual building because details of connections and restraint may be unlike those of the test.

In addition to the effects resulting from direct fire exposure of specific members, stresses may also develop in unexposed parts of a building due to the movement and interaction of other members of the building that are exposed to the fire. All of these conditions increase the possibility of some structural damage in the fire. However, it should be recognized that the primary concern of building codes is life safety, and that limited structural damage without collapse should be acceptable. Existing fire test standards do recognize this point, but it is conceivable that damage in actual fires may be greater or less than in test fires because the conditions in a building are so very different than in the fire tests.

To limit the structural damage as much as possible there appears to be a need for the structural engineer to consider the effect of fire exposure in developing his design. In the long range such improved designs should result in safer buildings and safer structures of all types.

MR. GILLIGAN

Our next speaker will cover another aspect of this input: the mechanical and physical properties of structural metals at elevated temperatures. We are fortunate in having such a knowledgeable person on this subject -- Dr. George V. Smith. Dr. Smith received his Doctor of Science degree at Carnegie Institute of Technology, now known as Carnegie-Mellon. He was with U. S. Steel Corporation for 14 years at the Fundamental Research Laboratory then located at Kearny, New Jersey. Following this he was at Cornell University. Throughout all of these years George has specialized with elevated temperature properties and characteristics of metals. Last year George retired and is now engaged in consulting engineering.

DR. SMITH

I was told that this audience is principally interested in the effects of fire upon compression members, columns, buildings and bridges made of steel, and that all other aspects of the effects of temperature upon materials might be touched upon only very casually. If one accepts these premises, then it would seem that the effects of fire in determining modes of failure might include the following as the principal possibilities: plastic yielding on one hand and elastic buckling or collapse on the other hand. If this is true, then the material properties which are p-incipally

Panel Discussion

of interest to you are the following: yield strength, elastic moduli and thermal expansivity coefficient.

Fortunately, the elastic moduli and the expansion coefficient of steel are relatively insensitive to chemical composition and to heat treatment. We can, to a good approximation, conclude that these properties are independent of all the variables of composition and heat treatment that you might contemplate.

I have some slides which will illustrate how these properties vary with temperature. In the first slide (Fig. 1) the modulus of elasticity of steel is compared with that of a number of other common materials. You see evidence in this slide that the modulus of elasticity (the tensile modulus or Young's modulus) is insensitive to composition within rather wide limits, and in fact is really not very much different if one goes from a ferritic type of steel to the austenitic stainless type of steel. For all of these metals, the modulus of elasticity decreases rather slowly with temperature, certainly at the outset, and then perhaps tends to exhibit an acceleration of fall off as the temperature becomes higher. The shear modulus of elasticity shows a somewhat similar trend with temperature. Poisson's ratio remains essentially unchanged, to a first approximation. Perhaps I should have prefaced this illustration by pointing out that the effect of temperature on modulus of elasticity is of interest to you in relation to buckling in a very direct sense. But it is also of indirect importance because it is the proportionality constant by which differences in length arising from temperature differences are translated into stress, and hence into what we sometimes refer to as thermal stresses. With steel and its high modulus of elasticity at 30,000,000 p.s.i., one converts strains into stress at a pretty fast clip -- 30 p.s.i. for each 10^{-6} strain.

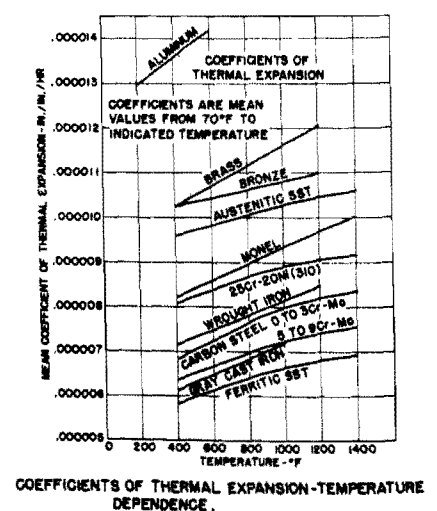
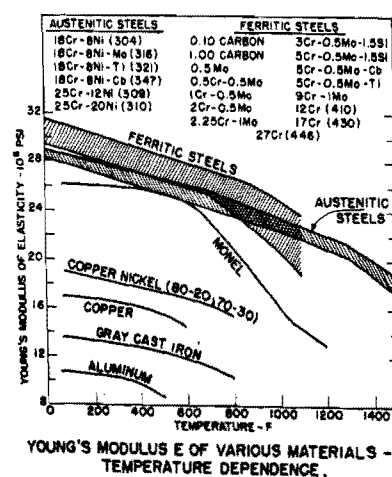


Fig. 1 Young's Modulus E of Various Materials - Temperature Dependence.

Fig. 2 Coefficients of Thermal Expansion - Temperature Dependence.

Panel Discussion

The second slide (Fig. 2) shows the dependence of expansivity coefficient of steel on temperature, in comparison with a number of other materials. This coefficient increases somewhat as the temperature increases. There are wide differences amongst materials, which in itself incidentally can lead to stress problems independent of those that might be evident in a homogeneous structure -- homogeneous in the sense of a similar material. If, for example, one combines a ferritic steel with an austenitic steel, with the rather marked differences in expansion coefficient, then even if there are no temperature differences one can have very significant stresses arise simply because these two materials choose to expand or contract at different rates as the temperature is changed. I suspect that the matter of the expansion due to temperature differences that develop, or temperature gradients within an individual member, or temperature differences between different portions of a structure that are originally joined to one another, is a rather significant aspect of your concern about fire effects.

Let us now turn to the third property that I mentioned, yield strength. Unlike the first two properties that we touched upon, yield strength is sensitive to chemical composition and heat treatment or prior processing. (See Fig. 3) We have evidence of considerable scatter when we explore the dependence of strength upon temperature. For example, even at room temperature the yield strength of A36 steel might range upwards from 36,000 p.s.i. to something on the order of 50% or more greater than this. So there is a rather large sensitivity to the variables of composition and to processing over and beyond those that are stipulated in the specifications themselves.

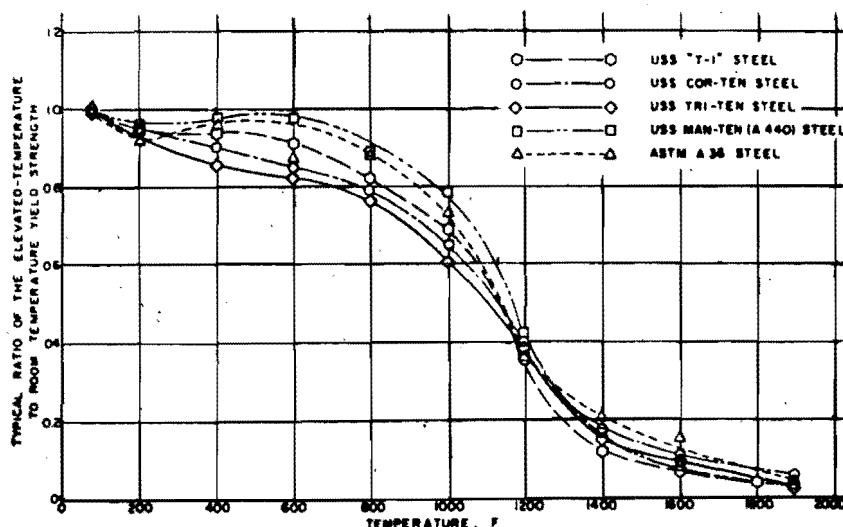


Fig. 3 Effect of Temperature on the Ratio Between Elevated - Temperature and Room-Temperature Yield Strengths.

Panel Discussion

To a first approximation, the strength at elevated temperature varies with the strength at room temperature; that is to say, a strong material at room temperature is likely to exhibit strength at elevated temperatures as well. It therefore becomes convenient to chart the variation of strength with temperature in the ratio form that is employed in the third slide, which compares the variation of the yield strength ratio with temperature for a number of materials. It is important to point out that the A36 steel has a minimum yield strength requirement of 36,000 p.s.i. whereas the T1 type of material has a yield strength requirement of 100,000 p.s.i. minimum. It is also of interest to note that the maximum scatter amongst these materials, in terms of deviation from the average ratio curve, is only on the order of plus or minus 10%. This is independent of the scatter in absolute values, which I will touch upon in a few moments.

The next slide (Fig. 4) shows the variation of the tensile strength ratio with temperature. I have introduced this slide in part to illustrate one of the aspects of steel behavior that is rather complex and difficult to treat, and that is what we refer to as dynamic strain aging. This dynamic strain aging to which some materials, steel in particular, may be subject, is manifested in an increased strength ratio, and also in absolute strength, as one goes from room temperature to 400 - 500°F. It is possible that the tensile strength is greater at 500 or 600°F than it is at room temperature. This is a manifestation of what we refer to as strain aging, which requires that plastic deformation is introduced and that time and temperature are provided. The higher the temperature the shorter the time that is required and vice versa.

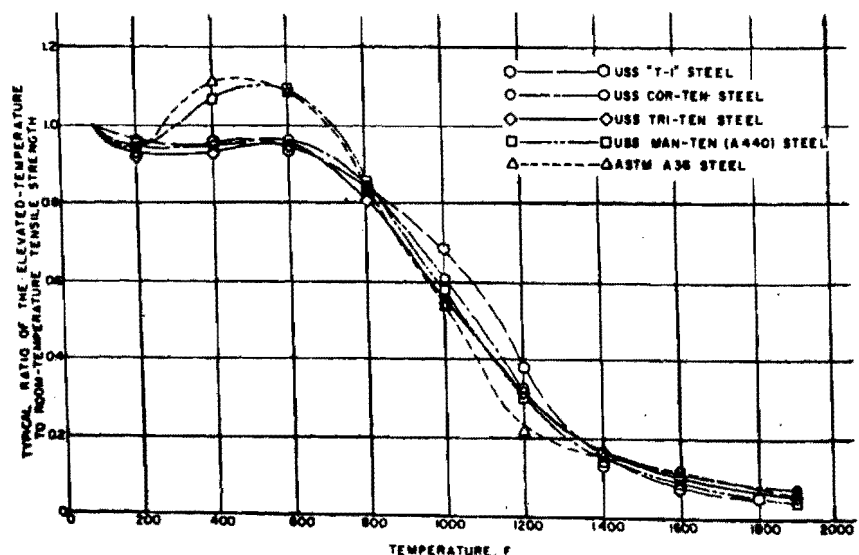


Fig. 4 Effect of Temperature on the Ratio Between Elevated - Temperature and Room - Temperature Tensile Strengths.

Panel Discussion

The plastic deformation that is required to set the stage for the dynamic strain aging is introduced in the early stages of the tension test. Therefore the strain aging can manifest itself at intermediate temperatures, where the times are appropriate for the temperatures that are involved. But because the stage must be set for plastic deformation, one would not ordinarily expect this strengthening effect to be manifested in the variation of yield strength with temperature, because obviously there is little or no deformation involved at the time the yield strength is attained. However, if one has introduced plastic deformation prior to commencing this exploration of the effect of temperature, for example by cold forming or straightening operations, the stage can be set, and one does find evidence for strain aging in the variation of yield strength with temperature. Strain aging and the basic susceptibility to strain aging are related to the manufacturing of the steel, and in particular to the deoxidation practice employed in the manufacture of the steel, and its effect upon the nitrogen content. These are matters, incidentally, that are not really stipulated in any sensible degree in specifications. One does find evidence that strain aging is less prominent in the higher strength low alloy steels than it is in the carbon steel, such as A36.

One feature that is concealed by the ratio type of plotting is the scatter that I alluded to a little earlier, this is shown in the next slide (Fig. 5). I have introduced this slide, which relates to tensile strength to illustrate in terms of absolute values the magnitude and differences that may be experienced. The slide encompasses a restricted sample in the sense that it relates to carbon steel which has been deoxidized in the so-called course grain deoxidation process, and tempering or stress relieving has not been employed. The scatter ranges from some 60,000 to 85,000 p.s.i. at room temperature. This is not as

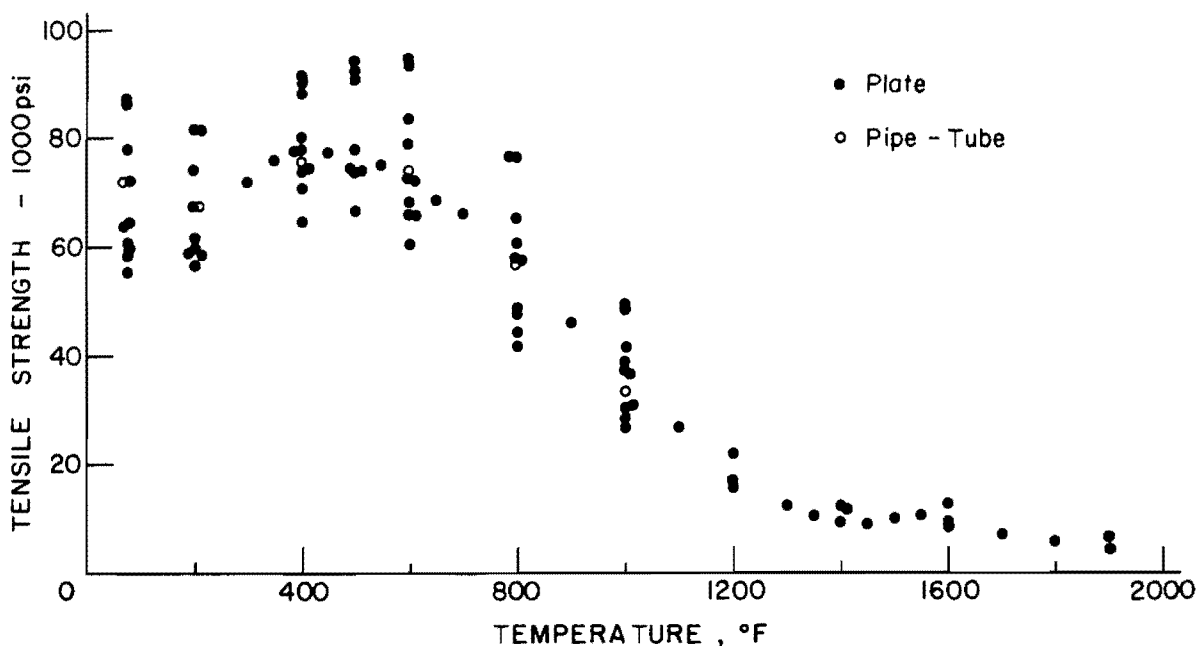


Fig. 5 Scatter of Tensile Strengths at Room-Temperature and Elevated-Temperature.

Panel Discussion

much as would be evident for yield strength, but nevertheless it is a rather significant scatter. One might argue that, since the allowable stress is based upon a minimum requirement, this is all for the good. However, it is true that if one goes beyond what is minimally required, there is a tendency for loss in fracture toughness. The extent to which you are interested in fracture toughness I can't really surmise. If loading is compressive, then perhaps this is a rather academic question. On the other hand, it does seem to me that under fire conditions, even though the initial loading is compressive, one might find the stress in some members reverse to tension, as a consequence of thermal stresses that have developed owing to uneven heating. There is a possibility, then, that there might be brittle behavior if one has strengths considerably higher than the minimum requirement.

I have been led to believe that you are not much interested in the creep phenomenon. However, you should be aware that if you go up sufficiently high in temperature, straining continues with time, that is creep occurs. In ordinary boiler and pressure vessel construction to the ASME code construction rules, the creep criteria may govern allowable stresses for steel at a temperature beginning on the order of 750°F. On the other hand, with the transient situations, with which you are concerned, I rather doubt that you might be troubled by creep until you reached temperatures exceeding at least 1,000°F. If this is not true, I will be happy to explore during the discussion period this question of creep. This is not to say, I should emphasize, that creep could not occur at lower temperatures than 750°F; in fact, some of you may be aware that steel can creep to a significant extent even at room temperature. Those of you that may be involved in pre-stressed concrete applications should be aware that the relaxation associated with creep that can occur in steel at room temperature may be significant.

MR. GILLIGAN

Getting a little closer to the problem now, our next speaker will direct remarks toward practical engineering problems. The speaker is Anthony F. Nassetta. Tony was graduated from City College of New York with a BS degree in Civil Engineering, and obtained an MS degree from New York University. Following graduation he was with the Corps of Engineers where he performed design functions on a number of military type structures. At the present time, and since 1946, he has been with the firm of Weiskopf and Pickworth. He is now in charge of major building projects for the firm. The list of buildings for which he has supervised the design is very impressive. He is a registered Professional Engineer in New York and other states in the East, a Fellow of the American Society of Civil Engineers, a member of the New York State Society of Professional Engineers, New York Association of Consulting Engineers, New York Building Congress, Consulting Engineers Council, American Institute of Steel Construction. He is the author of numerous articles and papers on engineering and architectural aspects of buildings, and he is co-author of the chapter, "Multi-Story Buildings" of the book Structural Engineering Handbook, edited by Gaylord and Gaylord. Mr. Nassetta serves on the New

Panel Discussions

York Building Code Committee, and is very active in Code work and in design for unique methods of fire protection.

MR. NASSETTA

I would like to talk to you tonight about the practical aspects of fire effects on structural stability, with special and particular emphasis on exposed steel type of buildings, that I have gotten familiar with recently.

The methods of analysis and design of structural systems to withstand the effects of wind, gravity, earthquake, temperature and blast are all known and accepted by the engineering profession. Fire effects on structural loads, assemblies and systems, on the other hand, are at present not subject to rational engineering analysis and design. As a result, all structural systems, depending on the size, height, and building occupancy, are required to be protected from fire effects by encasement in so-called fireproof materials. Furthermore, the ability of the fireproofing material or assembly to prevent critical temperature is established by the all-too-familiar standard fire test of building construction materials, commonly known as ASTM E119. Consequently, under present procedures, designing for fire effects becomes nothing more than establishing the required fire resistance rating and selecting the most suitable fireproofing material. The required fire resistance rating is generally established by Code, and fireproofing material is established by test.

In recent years, development of welding techniques in both shop and field, high yield point steel, weathering steel, and long-lasting paint systems have generated greater interest in architectural use of exposed structural steel. Several noteworthy buildings have been designed and constructed in this country and in Europe with exposed structural steel columns.

Could I have the first slide, please? I'm going to show you a few buildings, some in this country, some in Europe. This is the very famous and very handsome U. S. Steel building here in Pittsburgh. (See Fig. 6) I'm sure all of you have seen this building and are familiar with the fire protection system, particularly of the exterior columns.

Next slide, please. This one is considered, architecturally at least, the first successful all-exposed exterior column system. It's the John Deere building in Moline, Illinois. Also a very beautiful and architecturally excellent type of design.

Next slide, please. This is a building in Turin, Italy. It's the RAI building, and I think it's the radio and television center of northern Italy. If you look closely you'll see the columns and the girders are all exposed steel. I stumbled on this building and I was taken very much by

Panel Discussion

surprise to see this kind of design in Italy.

Next one, please. This is a university building in Turin. You can see the columns are entirely exposed all the way up the building. It's an eight story building, also very attractive and very modern.

Next, please. Another building in Milan, Italy, exposed steel columns.

Next, please. This one is also in Milan, Italy. This is the Chase Bank Building in Milan.

Next, please. I included this slide of the Eiffel Tower to show you and to remind you that exposed steel is not just a ten year or recent innovation. We've had it around for some time, but we've never really considered it in the same light as we have recently. The structure is very attractive, very well known, and believe it or not there isn't a single piece or element in the structure that's protected in any way with fireproofing material.

The next group of slides I am going to show you are some of the attempts that were made in designing the United States Steel office building in New York City, a 54 story building which is now almost completed. These are some of the prototypes that were studied. You can see that the attempt here was to express steel, particularly in the exterior and particularly in the very strong expression, very similar to the Eiffel Tower.

Next, please. Here again another attempt. This is a very beautiful concept, and I think if we had a little bit more time and a little bit more nerve, I guess, we might have carried this one off.

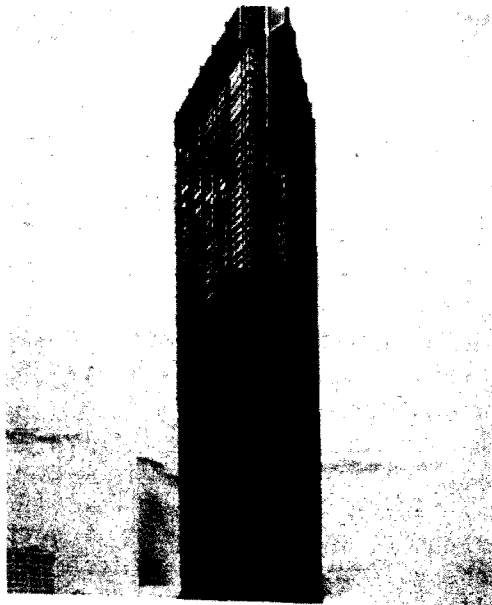


Fig. 6 U. S. Steel Building,
Pittsburgh.

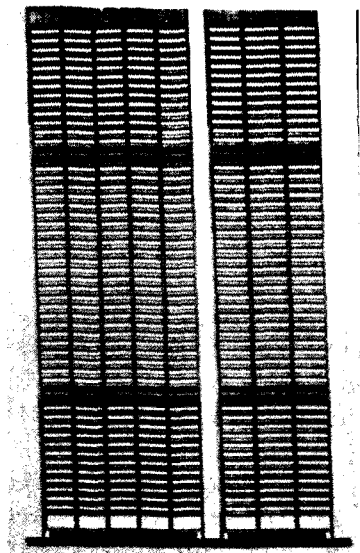


Fig. 7 U. S. Steel Building,
New York.

Panel Discussion

Next, please. This is a very visionary one which someday I think will be a reality. This is the exposed steel plate concept of design where the exterior wall consists of stiffened plates with openings for windows, fire canopies over all the windows, and protected only on the interior with conventional fire-proofing materials.

Next, please. (See Fig. 7). This is the prototype that was selected for the final design. It does express steel. It's very bold. We adopted this one largely because of the many meetings we had with Larry Seigel and the Applied Research Laboratories wherein this seemed to be the one that offered the most promise in being able to convince the Building Department authorities. We could demonstrate with tests and with some rational thermodynamic analysis, that it is entirely feasible and safe for a building of this height in New York City. The design adopted for this 54 story building totally expresses the structure, with exposed steel plate girder spandrels spanning 54 feet and extending full depth between windows. The spandrel members consist of 70 in. deep steel built-up girders with metal cladding and window frames attached to the top and bottom flanges only, forming the entire exterior wall assembly. Columns are fully protected with spray fireproofing and are completely covered with metal cladding.

Next slide, please. (Fig. 8) I'll quickly describe it if you can't read it. Cladding top and bottom flange, window unit, concrete floor fill, steel cellular decking, steel spandrel girder, rigid insulation, sprayed on metal fiber fireproofing, suspended ceiling,

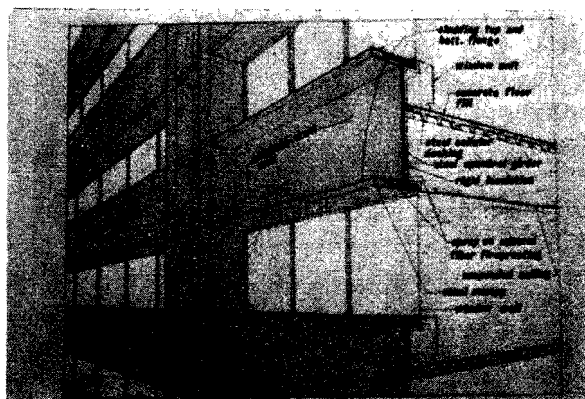


Fig. 8 U. S. Steel Building,
New York.

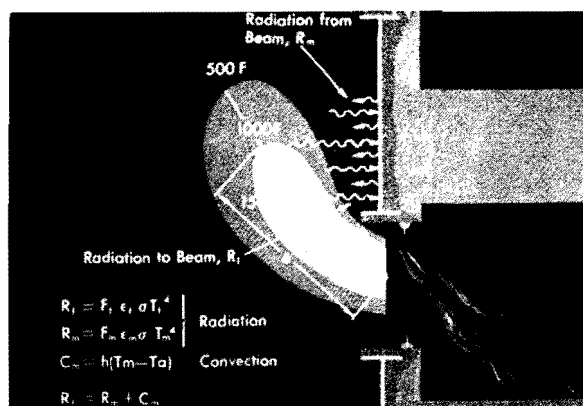


Fig. 9 Analysis of Heat Transfer for a
Spandrel Girder Under Fire
Exposure.

Panel Discussion

steel canopy, window, window wall, and, of course, that says exposed steel web and that writing at the extreme left is the column cladding. And this, basically, is the wall for that 54 story building. The design adopted for the 64 story U. S. Steel building in Pittsburgh also totally expresses the structure with exposed steel columns for the full height of the building. All other exterior members are conventionally fireproofed. The design adopted for a three story Shell Oil Data Center in Tulsa, Oklahoma, presently under construction, completely expresses the structure with all exterior columns and girders fully exposed for the entire building.

In each of these buildings different approaches for establishing structural stability of the exposed steel members under fire load were adopted. The 54 story New York City building employs flame shielding techniques; that steel canopy is one of the elements of the technique. The 64 story Pittsburgh building uses internal water cooling in the box columns. The three story Tulsa building has external water cooling of the exterior members. In each case, the principle of temperature attenuation without encasement was demonstrated by thermodynamic analysis and tests. Of particular significance is that much time, effort, and money were expended to obtain building department approvals of each of the above buildings.

I'd like to show you now some slides of the Trenton test on the U. S. Steel spandrel girder. The next slide (Fig. 9) was prepared when we were trying to demonstrate, at least explain, the analysis to the building departments. The explanation is very easy to understand. When a flame emerges from a building that's on fire, there is a heat transfer. Heat is radiated and re-radiated, and there is a balance. The analysis leads to that equation shown at the bottom of the picture, and it states that the system is in equilibrium. The equation predicts the temperature of steel under this kind of a fire exposure. This is the way flames emerge from a building.

Next slide, please. This is a series of curves which describe a particular fire or flame. These curves predict the temperature of the web of the steel spandrel girder under different flame temperatures.

Next slide, please. This slide shows the flame temperatures that are attained along the flame axis, depending on different fire loads. By the way, fire load is nothing more than the weight per square foot of combustible material in a given building.

Next slide, please. (Fig. 10) This is the Trenton mock-up. Again, we had to do a little bit of persuading here. The wall is a mock-up for architectural purposes. The architect wanted to study the scale, the texture, the effectiveness of the cladding and also the expression of the steel. When it served its purpose Larry Seigel said, "Let's burn it up", and he devised a special chamber. The roof of the chamber is roughly at the level of a typical floor of the building. So in effect we had a full scale chamber, which hopefully could be used as a way of persuading the building department officials that this was a

Panel Discussion

valid thermodynamic analysis. This analysis predicts less than critical temperatures under a fire load of 6 pounds per square foot in the building, which is considered to be, at the moment, a reasonable fire load.

Next slide, please. Various parts of the girder and column cladding were instrumented, and this picture is simply to identify the locations of the thermocouples.

Next, please. This again is inside the chamber where the thermocouples were placed so that we could describe the time-temperature curve and see how it compared with the standard time-temperature curve.

Next slide, please. This is an inside view of the chamber full of the wood cribs which were used to create the fire.

Next slide, please. Here is the fire. It was a hot fire. Unfortunately, it burned for only for 15 minutes and then it started to die down. This created a problem with the Building Department. Their one comment was that the fire did not burn long enough. So, although it was a big success to us, it did not give the desired results.

Next slide. This shows the building after the fire.

Next slide. (Fig. 11) This is the extent of the damage to the fire canopy that could be observed, but not the steel.

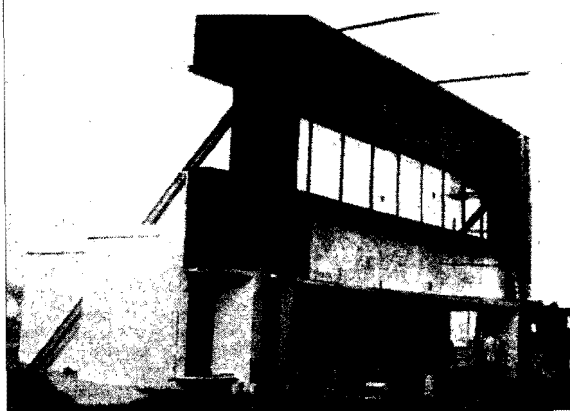


Fig. 10

Fig. 11

Fire Test on U. S. Steel Spandrel Girder

Panel Discussion

Next, please. (Fig. 12) These are a series of curves showing the measured temperatures on the web, and also the time-temperature curves that were actually obtained in the chamber. For comparison, the standard time-temperature curve of ASTM E119 is also shown. The highest temperature was reached in 15 minutes, and then it died off. After 40 minutes the fire was to all intents and purposes no longer a fire. Of equal importance is that the web temperatures never exceeded 600°F.

Next, please. This shows the temperatures of the cladding on the top and bottom flanges. Here one would expect higher temperatures, but of course the flanges were protected and the cladding was nothing more than a deflector or a canopy, making the flames emerge from the building without impinging on the web of the exposed steel girder. Here the temperatures get close to critical. As you can see from the slide, the fire did damage the cladding.

Next, please. (Fig. 13) These are the column temperatures. Surprisingly, and this is one of the things that we are trying to develop further, the cladding temperatures never exceeded 800°F. There of course we are fireproofing in the conventional way, and based on this curve, we are protecting the columns unnecessarily.

Next, please. This is a picture of the model of the building which is nearing completion in downtown New York City. I should add that two more tests were conducted for the benefit of the New York City Building Department, at Northrup, Illinois, and at UL Testing Laboratories. The fire department had to be convinced that during a fire lasting four, five, or six hours, the web would not reach critical temperature. In the test the

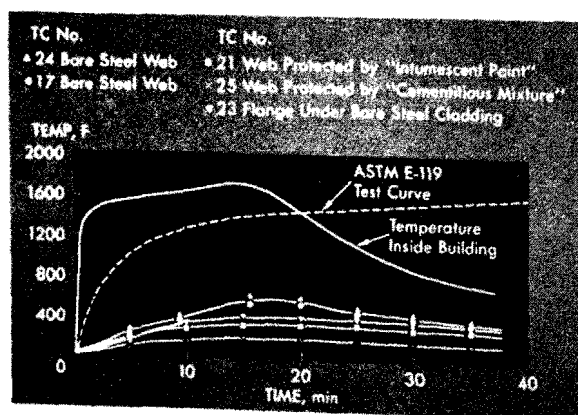


Fig. 12 Time-Temperature Curves for the Web.

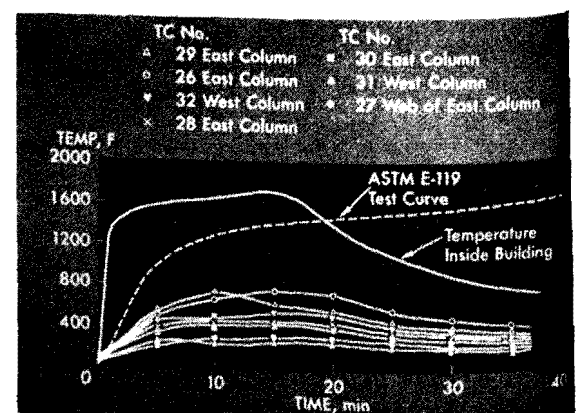


Fig. 13 Time-Temperature Curves for the Columns.

Panel Discussion

fire lasted 3 1/2 hours, I think. We just ran out of gas.

Well, there is no question that the prospect of special testing programs and time consuming presentations to building department deters many architects and engineers from proposing exposed steel designs. Furthermore, recent fires in high-rise structures have raised doubts about long accepted fire protection practices, including UL ratings, sprayed-on fireproofing materials and methods, and the standard time-temperature curve. The engineering profession must develop safe and proper procedures based on engineering principles when designing for fire effects. A design approach to fire effects based on engineering principles, using thermodynamic analyses for estimating temperatures of structural members, and mechanical and physical properties of metal at elevated temperatures, is long overdue.

MR. GILLIGAN

The official title of the next topic is "What can and should be done in areas of structural engineering research". The speaker is Richard W. Bletzacher, Associate Professor of Civil Engineering, Ohio State University. Dick received his Bachelor of Civil Engineering degree from Ohio State University and the Master of Science degree from the same institution. He is director of the Building Research Laboratory at the University and has been in this position since 1958. He is chairman of ASTM Committee E6 on performance of building construction, and past officer of ASTM E5 on fire tests of building construction. He is chairman of the ASCE Structural Division's Task Group on fire protection, a member of the National Fire Protection Association committee on fire test methods, and statutory consultant to the Ohio Board of Building Standards.

PROF. BLETZACHER

Thank you, John. As the only member from the academic community of the panel, I presume it was me that you were talking about saying I needed three months to go to ten minutes, because all of my lectures are rigged for 48. I can usually figure how to get them down to 30 by just talking faster, but 10 is ridiculous. I'm not real sure that this organization that you say you didn't have isn't working out fairly well at that. My only comment to Larry about those designers that only use UL, I notice that they also only specify Phoenix steel, so if you can stand it I can.

What I'd like to talk to you about is, number one, some context or concept of how building assemblies, that is, construction assemblies, structural assemblies operate, either in the test or in real fires in buildings, and then suggest to you an area of research that I think might evolve into an effective design procedure and could, with some luck, relegate this problem to essentially the same situation that we have with structural design.

Panel Discussion

The first point that I need to make is that the performance of building elements such as walls or partitions, floors, roofs, or whatever, subjected to an unwanted fire, really depends upon three distinct and separable characteristics in construction assemblies. I'm calling them separable, some of my friends in the fire protection field are not as perceptive, and in the standard fire test for fire resistance of these elements, ASTM designation E119, these characteristics are actually measured against two criteria. One criterion deals with the thermodynamic transfer of heat through the material membrane from the fire exposed to the unexposed surface of the assembly. This is thermal transmission all the way through, and in effect, letting the fire pass through the assembly to the adjoining space. We are dealing here with fire resistance and I am attempting to use these assemblies to confine the fire to the area of initiation. These membranes could be brick, or block or plaster, or gypsum wallboard, if we're talking about walls in particular, or ceiling tile or plaster or floor slabs. I wasn't going to use the word "flungdung", but there are some that have already called it fireproofing, and if you're going to call it fireproofing I'm going to call it "flungdung". We're really talking about spray applied fire insulation materials.

The second criterion in the test method, no question about this, deals with the structural integrity of the construction assembly. I'm using structural integrity in a little broader sense than Larry did, because I'm talking about a construction assembly, not a single structural element. It involves the protection of these structural elements from either critical thermal stress buildup or temperature deterioration of the material which constitutes these load carrying members. In some instances the membrane resisting this thermal heat transfer has dual function, such as a concrete floor slab, wherein it's both the thermal resistance membrane and the structural element. These two criteria of acceptance used in the test deal directly with two of the characteristics in the construction assembly itself; that is, the thermal protection and the structural integrity.

There is another characteristic, though, that third characteristic in the assemblies that is critical, or can be critical, and deals with what I will call the premature failure of these protective membranes, whether it be the flungdung on the steel or the ceiling membrane protecting the structure from these elevated temperatures, or it's those membranes protecting the thermal transfer all the way through the assembly. I'm talking about this as premature failure of the attachment system, or structural weakening, failure to support membranes such as gypsum wallboard or whatever, such that as they calcine, decompose or otherwise deteriorate from the exposure to temperature, they prematurely fall off and no longer are present for the protection process. When that happens the test actually picks it up either in the criteria for thermal transmission or in the criteria for structural integrity. But in the test we don't really measure that particular phenomenon, we only observe it. The prediction of thermal transfer is tractable to analysis. The possibility of predicting the structural integrity is also tractable to analysis, but the prediction of the premature falloff is not tractable. Therefore, we may not be able to totally do away with the test. What does bother me is that this phenomenon which confuses the results, or messes up the results that we might be able to predict by analytical procedures, cause some to say that none of analytical procedures are any good. We've had three problems. If we can start solving two we can worry about that third one next, and we

Panel Discussion

shouldn't let that stop the advancements in the other directions.

I will now talk about a proposal which we prepared some time ago in cooperation with Buford Gatewood who is a professor of aeronautical astronautical engineering at OSU. Professor Gatewood is nationally recognized as an expert in thermal stress analysis and has a textbook on the subject, and has had extensive experience utilizing this in analysis of airframe structures subjected to elevated temperatures. My personal experience in airplanes is on the inside - and not even in the cockpit, so I was not aware that at this minus 65° F temperature at high altitudes the friction even through this thin air does build up temperatures on the surface of aircraft and imposes thermal stresses that I have not contemplated. Prof. Gatewood has had the opportunity to work on this, and techniques would be adaptable to our problems in building frames.

He published a paper in Journal of the Structural Division in April of '65 entitled "Tridiagonal Matrix Method for Complex Structures". The technique outlined there is tractable to handling a complete building frame. The computer program Professor Gatewood developed out of this can accommodate, we believe, a four story frame with up to three bays by five bays, and it uses the structural elements in the building frame as beam members. Each of these members can have three load components and three moment components, and each joint can have three deflection components and three rotation components. The individual beams can have variable areas, variable loads, variable temperatures, variable material properties, inelastic materials and elastic materials, provided the effects of those variations are converted into input endloads and moments on the member.

Professor Gatewood has also been involved in another study at North American Aviation, which developed a second program for use on the tail assemblies of airplanes. This program can convert the variables for an individual beam into the requisite input endloads and moments for the first program. The second program then deals with the small element and the first program takes the output of that as the input to loads of the surrounding frame. In this sense we could deal with the variable load, variable material properties, and variable temperatures as inputs to the second program. The output of that would then be used as the input to the first program which then would distribute that over the whole building frame. It would seem to me that these two programs in tandem would permit us to predict the structural performance against appropriate failure criteria. criteria.

Before you sense I've gone overboard, I don't want to imply that we have the solution at hand. We have a number of aspects that need to be investigated, both for analytical and experimental verification. As a matter of fact, we've identified or defined some 18 tasks that need to be performed. If we assume that we have the variations in material properties as the function of temperature pretty well in hand we have to worry about the interaction of the beam and slab, and the beam, slab and girder and frame at elevated temperatures. These sort of interactions have to be determined.

In a real building we're going to have to do something about "lumping" these members. If I can characterize a real building frame and concrete slab and some joist and suspension system underneath, what I am

Panel Discussion

suggesting is that we have to convert those into some kind of equivalent beams and trusses to reduce the number of structural elements. When you consider the number of slabs, beams, girders, studs, joists and column elements in a building it's obvious that we're going to have to reduce the number by some approximating process to get it to manageable levels for a computer program, and we have to validate these approximations. The coupling of these two programs - the output-input interaction - has not yet been attempted so this has to be validated. The ultimate objective that we're looking for is the development of a design methodology. The range of thermal loadings, not really unlike the design live loadings used in structural design, would have to be established.

We need such a design procedure and we need this coupled with a companion technique for the thermal resistance. Thank you.

MR. GILLIGAN

If the panel will assemble up here we will get into some questions and answers. While they're assembling, Dick, you haven't shocked us at all I mean, we're used to handling umpteen variables and all things working at one time, and perhaps the reason why we're having this little session is we've run out of new variables so we're just looking for some more input.

Gentlemen, as I mentioned we're trying to record this, so when you address a question please speak up. If you'll identify yourself, if we don't think it's being recorded I will try to repeat the question and see how we make out with the panel. Who'll be first?

REIDAR BJORHOVDE

In the presence of the experts, I find it rather peculiar that the term "fire load" was not given more emphasis. In my opinion it is one of the most significant factors in fire protection and fire stability. To give a brief definition: The fire load of a structure is defined as the heating energy of all the combustible materials in the building per unit floor area. It is usually expressed in units of kcal/m^2 (BTU/sq.ft.), or converted into an equivalent weight of wood per unit area.

There are a few things I would like to mention in general reference to the topic of the discussion. As far as the code is concerned, I believe that all buildings ought to be classified in terms of their fire load, what use they are intended for, the type of the buildings, neighborhood, and so on. About ten years ago, the European Convention of Constructional Steelworks investigated the actual magnitudes of the fire loads in various buildings. Of particular interest are the results for office and apartment buildings, where the fire load was found to be extremely low.

Furthermore, the classification of materials and protection systems must be bound together with the fire load, because the magnitude of the load, together with a knowledge of the kind of combustible materials that are present, have the utmost influence on the way a fire will develop its duration, and the temperature that is attained. This has been illustrated by Mr. Nassetta.

Panel Discussion

The standard fire test can not be termed a rational basis for the evaluation of protection system characteristics. Rationale and economy can only be achieved by taking into account also the duration of the fire and the temperatures attained. An example might illustrate some of the points I am trying to make: a fire load of 25 kilos per square meter (5.5 lb/sq.ft.) will lead to a fire duration of about 15 minutes, with a maximum temperature of approximately 700°C (1300°F), which is maintained only for a very short time. These data come from a Swiss classification system, and I might mention that a building with fire load characteristics like these will be required to have the least amount of protection. The results from the investigation of actual fire loads in buildings that I mentioned previously thus illustrate why office and apartment buildings require relatively little fire protection.

The advantages and possibilities of using exposed structural steel elements in a building were forcefully demonstrated by Mr. Nassetta. This principle is far from novel, however; - buildings with, for example, exposed exterior columns have been used for many years in Germany and Switzerland.

Fire protection materials like gypsum, asbesto-cements, vermiculite and so on, have proved themselves highly efficient in a number of cases. (I am concerned here with lightweight encasement of structural elements; not the sprayed-on fireproofing, that actually has been shown to perform relatively badly in a fire.) An example of what this may mean to such an important factor as the dead load of a building - and this becomes increasingly important for tall buildings - may be of interest. In a 26 - story office building in Germany, two systems of fireproofing were designed, both fulfilling the requirements of the code. One of the solutions employed complete concrete encasement of the elements, whereas the other one utilized lightweight encasement of the type that I mentioned. The weight of the fireproofing materials only, was 3850 metric tons (85000 kips) in the first case but only 646 metric tons (14200 kips) in the second case. These figures speak for themselves.

Another point raised very often is: "We have to prevent a fire from starting". I would say immediately that this presents an impossible task, and rather emphasize the need to limit the extent of a fire, when and wherever it starts. Many of the disastrous fires that have occurred, especially in large buildings, could have been kept to a "reasonable" volume by making extensive use of partitions, water-"skirts", and the like.

In conclusion, there are a number of systems that can be used to evaluate the fire protection needs of a building. Of main interest are those that are easy to use for the practicing engineer, that do not demand too much time; since it would not make much sense to spend as much time on the design of the fireproofing, as on the structural design itself. One of these systems, that especially deserves mentioning, is the so-called Point Classification System. It has been used in Italy and Yugoslavia quite some time, and has proved itself versatile, economical, and rational.

MR. SEIGEL

I suppose really that fire loads in detail are not of extreme

Panel Discussion

interest to this group. At least it was my assumption that structural problems are of most concern. It is the fire load, without question, that causes the fire. Fire intensity and duration ultimately results in some kind of a temperature development on the steel member or whatever the structural member is, and therefore that is the cause of the structural problem.

Now, certainly there is need to know about fire loads and by no means has it been neglected in this country or in any European country. In this country there was a survey conducted many, many years ago. It was reported in BMS92, which is a publication of the Bureau of Standards. One of those issues was in 1942. It was revised in 1949, and a newer edition was published in 1970. All of these publications are fire load surveys of the combustible content of buildings in which we do have a pretty good handle of what is in a building.

As far as particular situations are concerned, before the fire test was run in Trenton, the one that Tony Nassetta showed on the screen, we surveyed a 40 story office building in Pittsburgh to find out what a bunch of pack rats that had been in the building for a long time might accumulate. The average was less than 5 lbs. per square foot. This was the basis of the test that was run in Trenton. Since there was an exposed steel member involved, and because we realized that the heat transfer to the steel would be very rapid because it had no thermal protection, we wanted to get the largest, most radiant, hottest flames that could be developed. That was the reason for selecting the fire load in the form that it occurred.

The last speaker mentioned the duration of the fire, and this is a very difficult thing to approach. It cannot be approached simply on the basis of pounds of combustible material per square foot, or BTU per unit of area. It depends on so many factors, including the amount of ventilation present, and the form of the fire load. For example, 5 pounds of toothpicks per square foot is a lot different than 5 pounds of railroad ties per square foot, and so obviously fires burn for different durations depending on the geometry of the fire load.

I would only say further that studies of fire development are in progress now in the Federal Construction Council. They are studying the fire growth, as it develops, from one part of a space to another part of the space, and then from that space to other spaces in the building. They have done this on the basis of probability, starting with what is called a work station. For example, this might be a work station, but first they discussed the probability of something like this paper igniting and then the work station is a more intense ignition source for something else. Will it ignite the lectern, and then will the lectern ignite the table, and will the fire take off? So this whole process of fire growth is being studied. And there are at least five countries that have developed methods of calculating rates of burning, or time-temperature curves that will be developed in buildings. To show how significant this is with regard to inconsistency between requirements and actuality, for our building in Pittsburgh the requirements for the columns are four hours of fire resistance and we designed them for that because that's what the Code says. Calculating on the basis of the Japanese method and one that we've developed in this country, the probable maximum duration of a fire in that

Panel Discussion

building turned out to be 45 minutes. So you see there is no relationship between what may happen and what is required. The Codes have been based on a very great safety factor, and there are misconceptions about the idea that a building which has a longer fire resistance is likely to be a safer building. That misconception is gradually being dispelled, particularly because of the recent fires in New York City where the deaths have been attributed to smoke. If you review history you will find almost all the deaths have been attributed to smoke. Not so many people really burn to death. They get away from the heat but they get trapped in the smoke. So smoke is being recognized as the real culprit.

MR. R. R. GRAHAM, U. S. Steel Corporation

I want to ask a question of Tony Nassetta since he brought up the fires in New York...whether or not any structural damage ensued. I think there have been three fires lately in high-rise buildings. Was the fire protection adequate and what type of thinking has the Building Commissioner, or whoever is responsible, adopted with regard to possible changes in the Code?

MR. NASSETTA

The one that I am most familiar with is the Plaza fire. In that fire, which occurred on the 33rd floor and spread to the 34th floor and damaged the 35th floor, the structural damage was confined to one beam connection that I remember. The filler beams in several locations were twisted, and one girder was twisted. A lot of deck was badly warped and needed replacement, of course affecting the topping on the floor above. There was no column damage. Basically the structure withstood the effects of the fire in the way the fire tests predict, and the type of structural damage was the type that you would expect in a hot fire. Nothing collapsed, nobody was hurt or killed, and the building itself was not in danger of falling down because of this fire that occurred on these floors.

The second part of the question is tough to answer because here you are dealing with people and emotions, and it becomes over-reacting. At the moment the biggest problem is the spray fireproofing and its suitability --- the way it is applied, and the disregard of other trades of workmen to the material and its function in the building. This is causing great concern not only to the building officials requiring controlled inspection of this construction procedure and material, but also to the fire rating agencies and the underwriters. The rating organizations are considering sprayed fireproofing as exposed steel, unless you can present a well-documented deal. I have seven items here, on which I have to start collecting data in order to overcome this new dilemma that we have on the buildings protected with spray fireproofing.

As far as the building officials in New York City are concerned, the question of deck fireproofing has become very, very acute. The deck manufacturers are also very much concerned about this. The H. H. Robertson approval of design 267, which has spray fireproofing on the underside of the deck, has a 2 hour rating, I believe in the unrestrained condition, according to UL. This deck has given the Commissioner and the deck

Panel Discussion

manufacturers so much concern and worry that there was a ruling yesterday to decide whether or not the ratings are valid but the Commissioner is still trying to add another requirement to the ASTM criteria of acceptability. So it has created a real box of snakes, and everybody is doing things to somehow prevent or overcome the apparent undesirable nature of this type of protection in these buildings that have been affected by fire.

I think the 919 3rd Ave. structure is similar to this. Structurally I thought it was a big success, but not the Fire Commissioner and not the fire rating agencies and not the press or public, and especially not the firemen. All of this is emotional and really not valid. It is beyond the engineering and the real structural aspects of the problem.

MR. J. B. SCALZI, U. S. Steel Corporation

It is my understanding that a column test for fire endurance is made with the fire completely surrounding the column. When exposed columns on the outside of a building are close to the curtain walls, the fire occurs on one side only and produces a temperature gradient across the member. What happens to the load carrying ability of the column in this case? How is it tested?

The other question is that of beam fixity and column restraints. I think these are problems that have to be considered somewhere along the line. I wonder what the panel might say about them.

Also, I would like to have a definition of "collapse" in terms of fire interpretation versus the definition from the standpoint of a hinge mechanism or yield point or excessive deflection. If the panel would answer the question of definition of "collapse" and then discuss the effect of a fire gradient across a column, and beam end fixities, I believe it would be helpful to the group.

PROF. BLETZACHER

So far as the test is concerned, collapse is when the assembly can no longer sustain the applied load. You are really not dealing with the plastic hinge mechanism and then the ductility range and on. Here is some load deflection data, and time-temperature deflection data on a series of beams that had a concrete slab 4" thick and 3' wide a 12WF27 steel beam with sprayed fire insulation. Some were tested simple span-no end restraint others were simple span but with end restraint generated as the test progressed, fixing degree of rotation, amount of axial expansion and this sort of thing. After the fire test starts you can see you're going to get some thermal stresses, you're getting some added bending, and finally at the end of the test you're getting precipitous deflection. We don't have any really flat rotation sort of thing, it just continues to go right on. That really is collapse, when it's continuing to deflect and can no longer sustain the load as it's being applied.

Panel Discussion

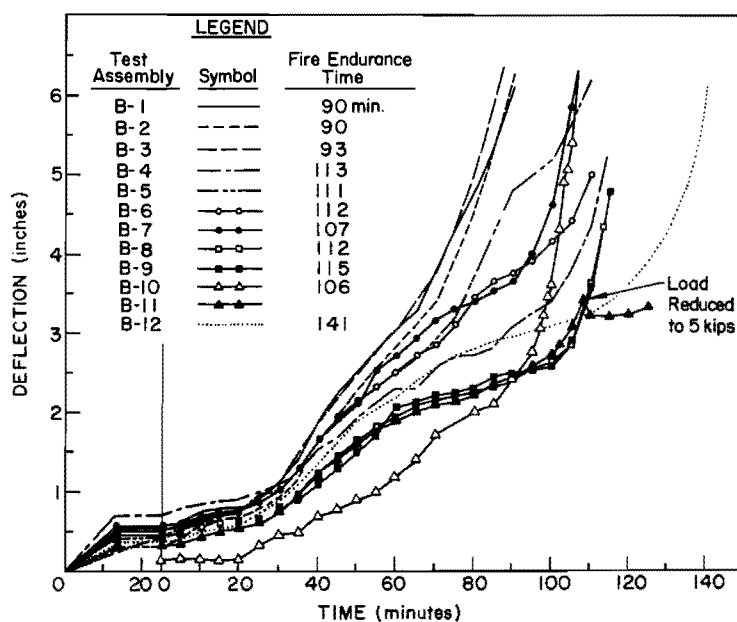


Fig. 14 Load and Time-Deflection Relationship.

MR. SCALZI

The figure is not too clear? What is the time scale on the figure? Where is the hour interval?

PROF. BLETZACHER

An hour is right here, 60 minutes. 120 minutes right there.

MR. SCALZI

What is the deflection at those time intervals?

PROF. BLETZACHER

The deflection was 6" at 90 minutes for the simple span unrestrained, and from 106 to 115 minutes for the restrained group, the beam with optional restraint went 141 minutes. In other words we were dealing with restraint, and the effect of restraint. Without restraint we had 90 minutes. With random restraint, not controlled restraint, I had an increase in fire resistance of 25%. When I add optimal restraint I had an increase of 50%.

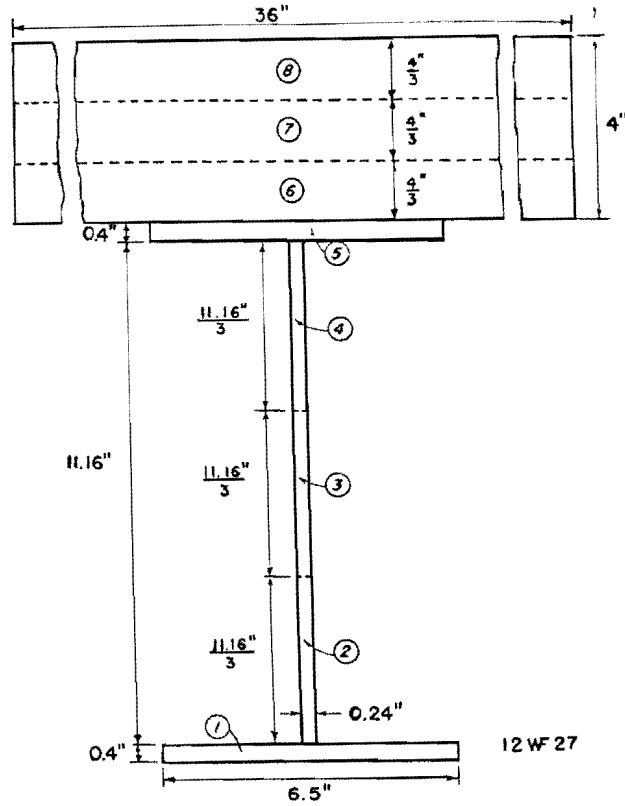
Panel Discussion

Fig. 15 Segmented Elements of Idealized Beam Cross-Section.

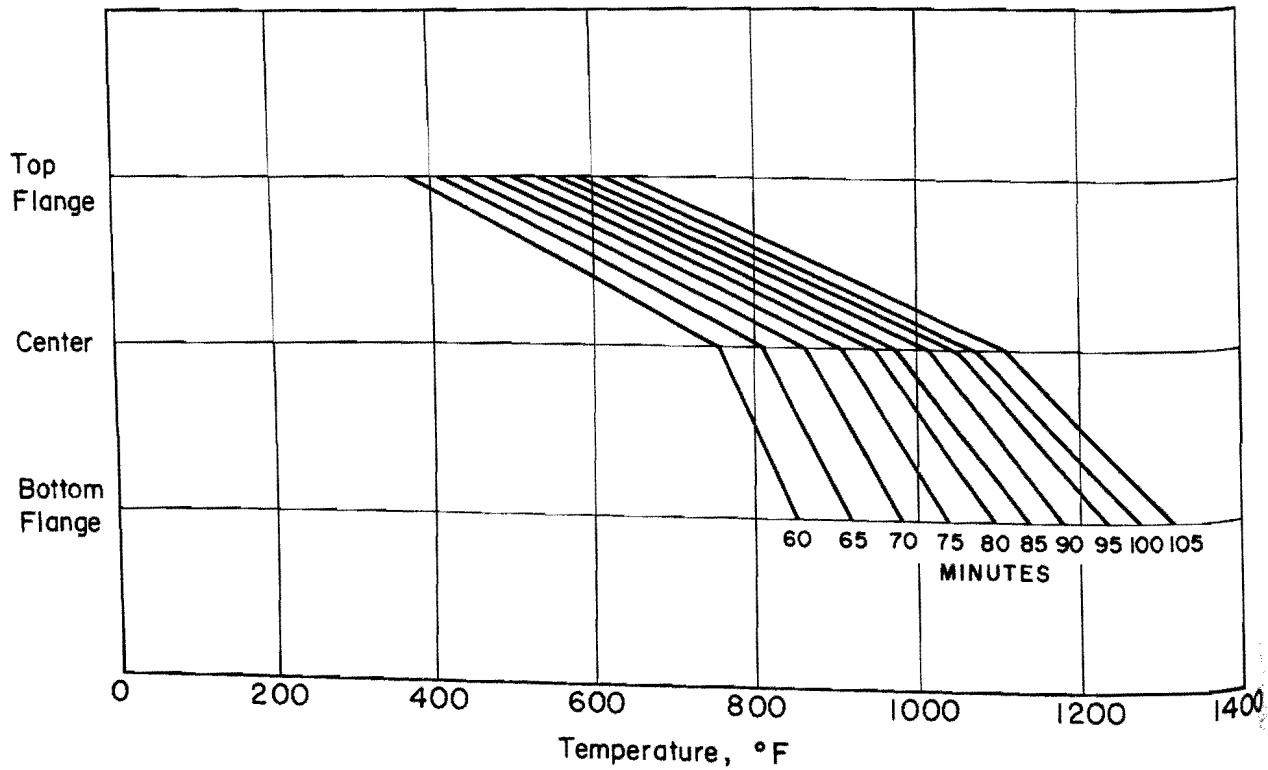


Fig. 16 Idealized Temperature Distribution on Cross-Section of Protected Steel Beams.

Panel DiscussionMR. GILLIGAN

I see we have a lot in common. I think his definition of collapse is similar to ours.

Jack Scalzi had another question related to a very interesting problem: the thermal gradient across a column. It could be any type of member. Who wants to talk about thermal gradients?

MR. SEIGEL

You might just like to have some numbers. Thermal gradients can be almost anything, depending upon the type of protection that is used for the structural member. If the member is bare there is likely to be a very high rate of heat transfer to the member, and under those conditions you can easily calculate what the temperature gradient would be by using the conduction equation, the conductivity of steel, the thickness and so on. For example, take the liquid filled columns in the U. S. Steel building in Pittsburgh in a four hour E119 fire. I am quite sure we are not going to have that fire, but assuming such a fire, the maximum surface temperature at the base of the column, where the plate is 4 in. thick, would be about 640° F. It starts at a relatively high temperature inside the column because the pressure on the liquid at the base of the column is fairly high and therefore boiling will not commence until the temperature is above 300°F. The temperature gradient within the steel itself in that case is 85°F per inch. The gradient is different on each side of the column. The face of the column that is away from the building would have a lesser fire exposure and as a result there would be lower temperature gradient over there.

As another example, there is a building in California that also has liquid filled columns of a box shape 18 in. X 12 in., fins extending from the outer surface. The temperature at the tips of those extensions can go as high as perhaps 1600°F, while at the root which is near the water in those particular columns, would be more in the order of 350°F. These temperatures are roughly equivalent to those commonly used for flame bending of bridge girders. The column may bend one way during the fire and after the fire it may take a permanent set in the other direction and there will be some damage to be repaired. So there will be some structural damage during a fire and I don't think it is realistic to expect this not to happen.

MR. GILLIGAN

Of course, at those temperatures one of our other great problems goes away. We worry a lot about our residual stresses. These stresses are washed away and the design becomes quite a bit simpler. Dick has some more on this same subject.

PROF. BLETZACHER

The point that Larry made though about the time at which these

Panel Discussion

temperatures are achieved is a function of the level or degree of protection that's on the assembly. In these particular tests we had 7/8" of sprayed fire insulation material on the contour of 12" wide flange 27 lb. beams. I expected them to last 90 minutes, that's how we had them designed. At 90 minutes when some of these first ones, unrestrained, started to fail, the bottom flange temperature was just a shade under 1200 degrees. The mid height of the web was just a shade over 1000, and a shade under 600 for the top flange, so this is a kind of a gradient and maybe not really unlike the situation of the column exposed on the inside and not on the outside the fire.

MR. GILLIGAN

George Smith left an open question which I think we ought to get settled. Is creep a problem in this fire problem from the standpoint of metal properties? Are we at temperatures, times, or conditions such that creep must be considered?

PROF. BLETZACHER

I kind of think it's a secondary effect.

MR. SEIGEL

The important thing to recognize here is the fire resistance rating and its meaning. What is required here by the building official, and how is fire resistance measured? We are interested in minutes to achieve a rating that is given in hours. As I tried to say earlier, if a test runs 58 minutes you lose everything, but if it runs 59 minutes they are willing to concede one minute and you win an hour. So under these conditions creep is important, because if creep, or lack of it, will provide an extra minute that is just what you need if you happen to be at 58 minutes right now. Therefore, I think creep is important in the sense of possibly winning a better rating during a test, but it is probably not very important in real performance in a fire because one or two extra minutes make no difference as far as people escaping from that building are concerned, nor is the ultimate damage reduced. So I don't really think that creep is important in connection with the temperatures that we normally expect to achieve.

As George Smith pointed out, as the temperatures increase the creep rates increase, and if there are temperatures of the order that Dick just showed, or those that I mentioned in connection with that building in California, then creep could be more important. But these conditions are unusual. In the building in California extra steel was put in the structure which, although it would get hot in a fire, would not have to carry load. Now, this is perhaps not a satisfactory way of thinking from the standpoint of the structural engineer, but if you're confronted with the problem of how to explain this to a building official who has a strict code to comply with you may be lucky if you can get the building up at all.

Panel Discussion

PROF. B. G. JOHNSTON, University of Arizona

I wonder if there is any comment on the actual failure of the McCormick Palace. How long did the fire last and did the fire heating the steel actually cause the failure or was there another cause?

MR. SEIGEL

That was an example of a misapplication of the occupancy. The Code permitted that building to be built without protection on those trusses because it was listed as an assembly occupancy. But on the night of the fire it was more like a mercantile occupancy because it had been rented for a housewares show, and so there was a huge fire load in the building. It really would not have made much difference if the fire load had been half that much, because the trusses were bare steel and they were down in a very short time.

MR. GILLIGAN

Gentlemen, time does not permit us to carry this interesting discussion any further. I thank the panelists, the questioners, and the audience for being with us this evening. The meeting is herewith adjourned.

ANNUAL BUSINESS MEETING

The Council holds an annual meeting for the purpose of reporting activities, election of members and officers, and presentation of the budget for the following year. The 1971 annual meeting was held in conjunction with the Annual Technical Sessions at the Pick-Roosevelt Hotel, Pittsburgh, Pennsylvania, on May 26, 1971.

The minutes of the 1971 Annual Meeting are as follows:

CALL TO ORDER

The meeting was called to order by the Chairman of the Council, Professor T. V. Galambos, at 10:45 a.m. Seventy members were present.

INTRODUCTION

The Chairman welcomed the members, and introduced himself, the Director, Dr. L. S. Beedle, and the Secretary, Dr. F. Van der Woude.

MINUTES

The Chairman presented the minutes of the 1970 Annual Meeting (March 25, 1970 at the Diplomat Hotel, St. Louis, Missouri) as printed on pp. 56-59 of the 1970 Proceedings of the Council. The motion that the minutes be approved (J.W. Clark/R.R. Graham) was carried.

REPORT OF ACTIVITIES

A report on the activities for the year 1970-71 was presented by the Director. The extent of the work of task groups, and of their findings, are indicated in the presentations made at the Annual Technical Session.

CRC GUIDE

The Director reported that the preparation of the third edition of the book "Guide to Design Criteria for Metal Compression Members", under the editorship of Dr. Bruce G. Johnston, is progressing satisfactorily. A detailed progress report was presented by Dr. Johnston at the Technical Sessions. Task group members were thanked for their contributions toward the revision of the Guide.

PARTICIPATING ORGANIZATIONS

A vote of thanks was extended to all participating organizations for their continued interest in the work of the Council. The Chairman welcomed Messrs. J. C. Simonis and M. P. Bernstein, the representatives from the American Society of Mechanical Engineers, and Dr. L. A. Boston, the representative from the American Petroleum Institute. The personal services and financial support of Mr. F. M. Masters were especially acknowledged.

FINANCIAL REPORT

A summary of the financial status of the Council was presented by the Director, including the proposed budget for the fiscal year 1971-72.

Budget Summary

Expected balance, October 1, 1971	\$13,000
Income	\$24,200
Expenditures	\$28,200
Expected balance, September 30, 1972	\$ 9,200

EXECUTIVE COMMITTEE

The Chairman invited Mr. R. L. Haenel, Chairman of the Nominating Committee, to present a report.

Mr. Haenel reported that the Nominating Committee had nominated Messrs. T. Dembie, J. A. Gilligan, and L. K. Irwin for re-election as members of the Executive Committee for the term 1971-74. The motion that the three nominees be re-elected (R.L. Haenel/R.R. Graham) was carried unanimously.

MEMBERS AT LARGE

The Chairman read that portion of the By-Laws pertaining to the election of Members at Large.

Since the 1970 Annual Meeting the following persons had been nominated by the Executive Committee:

Dr. A. Chajes
 Dr. W. F. Chen
 Dr. F. J. Lin
 Mr. F. J. Palmer
 Dr. S. U. Pillai
 Dr. S. S. Thomaides

In addition, Dr. B. G. Johnston moved, and Dr. L. S. Beedle seconded, the nomination of Professor T. Murray.

The motion that all seven nominees be elected as Member at Large (B.T. Yen/J.W. Clark) was carried unanimously.

NEXT ANNUAL MEETING

The Chairman announced that the next Annual Meeting of the Council will be held in Chicago, late March or early April 1972. The exact date will be announced later.

ADJOURNMENT

The Chairman expressed thanks to the members present, and in particular to Messrs. J. W. Clark, G. Haaijer and J. A. Gilligan for making the arrangements for the meeting.

The meeting adjourned at 11:10 a.m.

Respectfully submitted,



Frank Van der Woude,
Secretary

C R C C H R O N O L O G Y (1 9 7 0 - 7 1)

- 10Oct70 - Executive Committee met in Bethlehem, Pa.
- 24Nov70 - American Petroleum Institute joined the Council as a participating organization
- 15Dec70 - CRC Secretaryship changed hands (R. Bjorhovde to F. Van der Woude)
- 22Jan71 - Executive Committee met in Washington, D.C.
- 25,26Mar71 - CRC representatives participated in IABSE Colloquium on "Design of Plate and Box Girders for Ultimate Strength", held in London
- 25,26May71 - Annual Technical Sessions and Business meeting in Pittsburgh. 77 persons attended. 22 papers were presented
- 19,20Aug71 - CRC part-sponsored the "First Specialty Conference on Cold-Formed Steel Structures", held in Rolla, Mo.
- 7Sep71 - CRC Director, L. S. Beedle, met with the Japanese Column Research Committee in Tokyo
- 16-23Sep71 - Testing of heavy columns (H23x681, welded wide-flange section) at the National Bureau of Standards, Washington, D.C.
- 30Sep71 - Executive Committee met in Washington, D.C. and attended the dedication of the world's largest testing machine (12,000 KIPS capacity) at the National Bureau of Standards

T H I R D E D I T I O N O F C R C G U I D E

The CRC "Guide to Design Criteria for Metal Compression Members" includes background information and comprehensive design provisions. It is published as recommendations for specification - writing bodies and designers.

The second edition of the Guide was published in 1966. Preparation of the third edition was initiated in 1968 under the editorship of Dr. Bruce G. Johnston.

A detailed progress report was presented by Dr. Johnston at the Annual Meeting. The chart on the next page shows the present (September 30, 1971) status of the third edition.

All but a few chapters have been submitted to the editor, and final editing was started in September 1971. After approval by the Task Groups, Executive Committee, and Advisors the manuscript will be submitted to the publishers. Publication is anticipated in late 1972.

In addition to CRC funds, substantial support has been received from the National Science Foundation and from the American Institute of Steel Construction. Such support is gratefully acknowledged.

The continued dedication and skill of all co-workers, and particularly their efforts of the past year are very much appreciated.

~THIRD EDITION~ COLUMN RESEARCH COUNCIL GUIDE TO DESIGN CRITERIA FOR METAL COMP- RESSION MEMBERS		TASK GROUP & T.G. CHAIR- MAN	SOURCE & AUTH- ORSHIP	AS OF SEPT. '71			ED. DRAFT TO ADVIS- ORS & EX.COM.
T.G. MEET- INGS	PREL- IMINARY DRAFTS TO TASK GROUP	DRAFT TO ED- ITOR					
1	INTRODUCTION	~~~~~	PR.1,3,& ED.	~~~~~	~~~~~	→	5-22 '70
2	STRUCTURAL SAFETY		PR.1,2,3 & GALAMBOS	~~~~~	~~~~~	→	4-1-71 APRIL '71
3	CENTRALLY LOADED COLUMNS	T.G.1 GILLIGAN	PR.CH.2, T.G.1, TALL HOLT, & ED.	3-6-70 10-1-70	1-4-71 4-15-71	PARTIAL	
4	LOCAL BUCKLING OF PLATES	T.G.13 ERRERA	PR.CH.3 & T.G.13	3-23-70 5-24-71	4-27-71	AUG. '71	IN PROCESS
5	DYNAMIC LOAD EFFECTS	T.G.8 DA DEPPA	DA DEPPA	~~~~~	6-71	AUG. '71	IN PROCESS
6	LATERALLY UN- SUPPORTED BEAMS	T.G.15 HAENEL	PR.CH.4 ED.	~~~~~	~~~~~	→	APRIL '71
7	PLATE GIRDERS	T.G.16 F.D. SEARS (ACTING)	PR.CH.5 & T.G.16	6-8-71 9-17-71	PARTIAL SEPT. '71		
8	BEAM-COLUMNS	T.G.3-ELLIS+ 10-GALAMBOS	PR.CH.6+ PILLAI	12-12-69 11-23-70	PILLAI 9-24-70	12-23-70	APRIL '71
9	THIN-WALLED METAL CONSTRUCTION	T.G.13 ERRERA	PR.CH.3 & T.G.13	3-23-70 5-24-71			
10	TUBULAR MEMBERS	T.G.18 A.L. JOHNSON	CH.3+T.G.18+ CHAJES+ SHERMAN	5-26-71 8-17-71	CHAJES-8-10-70 T.G.18 DR. 8/71		
11	TAPERED STRUCT- URAL MEMBERS	T.G.7 AMIRIKIAN	LEE & T.G.7	12-18-70	~~~~~	→	2-8-71 MARCH '71
12	COLS. WITH LACING, BATTENS OR PERF. COVS.	~~~~~	PR.CH.3 & ED.	~~~~~	~~~~~	→	9-9 '70
13	MILL BUILDING COLUMNS		ED.	~~~~~	~~~~~	→	JULY '71
14	MEMBERS WITH ELASTIC LATERAL RESTRAINTS		PR.CH.7 & ED.	~~~~~	~~~~~	→	MARCH '71
15	MULTI-STORY FRAMES	T.G.4 IFFLAND	IFFLAND & T.G.4	6-26-69 12-8-69 9-30-70	IFFLAND 8-23-70 2ND-1-19-71	AUG. '71	AUG. '71
16	ARCHES	T.G.9 AUSTIN	AUSTIN & OJALVO	3-23-70	AUSTIN MAY '70 OJALVO OCT. '69	MAY '70	10-7 '70
17	STIFFENED PLATE CONSTRUCTION	T.G.19 KLINE	T.G.19	7-15-70 9-17-70	JUNE-1971	JULY '71	IN PROCESS
18	SHELLS AND SHELL LIKE STRUCTURES	T.G.17 BUCHERT	BUCHERT & T.G.17		1ST 9-5-69 2ND 12-29-69	10-20-70	11/70 REV. 8/70

A P P E N D I XProgram of Annual Technical SessionTuesday, May 25, 1971

8:15 a.m. - Registration

8:45 a.m. - Introduction
T. V. Galambos, Chairman, CRC

9:00 a.m. - MORNING SESSION

TASK GROUP REPORTS

Presiding: J. W. Clark, Aluminum Company of America

Task Group 1 - CENTRALLY LOADED COLUMNS

Chairman, J. A. Gilligan, United States Steel Corporation
"Maximum Column Strength and the Multiple Column Curve Concept"
R. Bjorhovde and L. Tall, Lehigh UniversityTask Group 3 - ULTIMATE STRENGTH OF COLUMNS WITH BIAXIALLY
ECCENTRIC LOAD

Acting Chairman, J. S. Springfield, Carruthers and Wallace, Ltd.

Task Group 4 - FRAME STABILITY AND EFFECTIVE COLUMN LENGTH

Chairman, J. S. B. Iffland, Praeger-Kavanagh-Waterbury
"The Sway Increment Method of Frame Analysis"
J. H. Daniels, Lehigh University
"Elastic Buckling Analysis of Space Frames"
S. Morino, Lehigh University

10:15 a.m. - BREAK

10:30 a.m. - Task Group 4 (continued)

"Stability Design of Steel Frames Under Combined Loads"

L. W. Lu, Lehigh University

"Stability of Braced Frames"

J. H. Davison, West Virginia University, and P. F. Adams,
University of Alberta

Task Group 7 - TAPERED MEMBERS

(Joint Task Group with WRC)

Chairman, A. Amirikian, U. S. Naval Facilities Engineering
Command

"Design Recommendations for Tapered Structural Members"

G. C. Lee, State University of New York at Buffalo

Task Group 8 - DYNAMIC INSTABILITY

Chairman, D. A. daDeppo, University of Arizona

"Comparative Studies of Unified Finite Element Techniques for
Dynamic Instability Analysis of Frameworks"

F. Y. Cheng, University of Missouri - Rolla

Task Group 9 - CURVED COMPRESSION MEMBERS

Chairman, W. J. Austin, Rice University

Task Group 10 - DESIGN OF Laterally UNSUPPORTED BEAM-COLUMNS

Chairman, T. V. Galambos, Washington University

"The Post-Buckling Behavior of Laterally Unsupported Beam-Columns"

L. C. Lim, LeMessurier Associates

12:00 p.m. - LUNCH

1:15 p.m. - AFTERNOON SESSION

TASK GROUP REPORTS

Presiding: G. Haaijer, United States Steel Corporation

Task Group 11 - EUROPEAN COLUMN STUDIES

Chairman, D. Sfintesco, CTICM, France

Vice-Chairman, W. A. Milek, Jr., AISC

"European Column Tests - Progress Report"

L. Tall and N. Tebedge, Lehigh University

Task Group 12 - MECHANICAL PROPERTIES OF STEEL IN INELASTIC RANGE

Chairman, G. F. Fox, Howard, Needles, Tammen & Bergendoff

Task Group 13 - THIN-WALLED METAL CONSTRUCTION

Chairman, S. J. Errera, Bethlehem Steel Corporation

"Structural Stability of Cold-Formed Steel Compression Members
Having Perforated Stiffened Elements"

W. W. Yu and C. S. Davis, University of Missouri - Rolla

"Impact Loading of Thin-Walled Cold-Formed Columns"

C. Culver, Carnegie-Mellon University

Task Group 14 - HORIZONTALLY CURVED GIRDERS

Chairman, C. F. Scheffey, U. S. Department of Transportation

Task Group 15 - Laterally Unsupported Beams

Chairman, R. L. Haenel, Pittsburgh Bridge & Iron Works

Task Group 16 - BUILT-UP GIRDERS

Chairman, F. D. Sears,

"Testing of Rectangular Model Box Girders"

J. A. Corrado and B. T. Yen, Lehigh University

"IABSE Colloquium on Design of Plate and Box Girders for Ultimate Strength"

J. W. Clark, Aluminum Company of America

3:00 p.m. - BREAK -

Task Group 17 - STABILITY OF SHELL-LIKE STRUCTURES

Chairman, K. P. Buchert, University of Missouri at Columbia

"Research Needs in Shell-Like Structures"

K. P. Buchert, University of Missouri at Columbia

"Dynamic Plasticity of Clamped Circular Plates"

D. Krojcinovic, Argonne National Laboratory

"Applications of Reticulated Hyperbolic Shells"

D. R. Sherman, University of Wisconsin at Milwaukee

Task Group 18 - TUBULAR MEMBERS

Chairman, A. L. Johnson, American Iron and Steel Institute

"Design Criteria for Structural Steel Pipe"

P. W. Marshall, Shell Oil Company

Task Group 19 - STIFFENED PLATE STRUCTURES

Chairman, R. G. Kline, United States Steel Corp.

4:30 p.m. - ADJOURN

8:00 p.m. - EVENING SESSION

Panel Discussion: "Fire Effects on Structural Stability"

Presiding: J. A. Gilligan, United States Steel Corp.

Panel Members:

L. G. Seigel, United States Steel Corp.

G. V. Smith, Consulting Engineer

A. F. Nassetta, Weiskopf and Pickworth

R. Bletzacher, Ohio State University

Wednesday, May 26, 1971

9:00 a.m. - MORNING SESSION

Presiding: T. V. Galambos, Chairman, CRC

TASK REPORTERS

Task Reporter 11 - STABILITY OF ALUMINUM STRUCTURAL MEMBERS

J. W. Clark, Aluminum Company of America

"Aluminum Members with Elastically Retrained Compression Flanges"

M. L. Sharp, Aluminum Company of America

Task Reporter 13 - LOCAL INELASTIC BUCKLING

Le-Wu Lu, Lehigh University

RESEARCH REPORTS

"Spaced Columns"

B. C. Johnston, University of Arizona

"Column Buckling at Elevated Temperature"

C. Culver, Carnegie-Mellon University

CRC GUIDE

Committee on the CRC Guide

Chairman, E. H. Gaylord, University of Illinois

"Progress Report on the Third Edition"

B. G. Johnston, Editor

10:30 a.m. - BREAK

11:00 a.m. - CRC ANNUAL BUSINESS MEETING

ANNUAL MEETING ATTENDANCE LIST

<u>Participant</u>	<u>Affiliation</u>
P. F. Adams	University of Alberta
W. J. Austin	Rice University
R. H. Batterman	Alcoa Research
L. S. Beedle	Lehigh University
M. D. Bernstein	Foster Wheeler Corporation
C. Birnstiel	New York University
R. Bjorhovde	Lehigh University
L. A. Boston	Cities Service Oil Company
G. M. Bove	American Institute of Steel Construction
K. P. Buchert	University of Missouri - Columbia
J. E. Campbell	University of Pittsburgh
A. Chajes	University of Massachusetts
F. Y. Cheng	University of Missouri - Rolla
J. W. Clark	Alcoa
C. G. Culver	Carnegie - Mellon University
J. H. Daniels	Lehigh University
C. S. Davis	University of Missouri - Rolla
J. H. Davison	West Virginia University
T. Dembie	Dominion Bridge Company
J. L. Durkee	Bethlehem Steel Corporation
N. W. Edwards	Pittsburgh - DesMoines Steel Company
W. E. Edwards	Bethlehem Steel Corporation
S. J. Errera	Bethlehem Steel Corporation
S. C. Fan	Bethlehem Steel Corporation
T. V. Galambos	Washington University, St. Louis
G. K. Gillan	Modjeski and Masters Corporation
J. A. Gilligan	United States Steel - Pittsburgh
M. I. Gilmor	Canadian Institute of Steel Construction
A. Gjelsvik	Columbia University
R. R. Graham	United States Steel Corporation
G. Haaijer	United States Steel Corporation
R. L. Haenel	Larsen and Ludwig
D. H. Hall	Bethlehem Steel Corporation
W. C. Hansell	Bethlehem Steel Corporation
A. J. Hartmann	Marquette University
M. Holt	Alcoa Research Labs
J. S. Iffland	Praeger Kavanagh Waterbury
L. K. Irwin	National Bureau of Standards
T. P. Jansen	McKaig Rupley Bahler
A. L. Johnson	American Iron and Steel Institute

<u>Participant</u>	<u>Affiliation</u>
B. G. Johnston	University of Arizona
R. G. Kline	United States Steel Corporation
K. H. Klippstein	United States Steel Corporation
D. Krajcinovic	Argonne National Laboratory
H. A. Krentz	Canadian Institute of Steel Construction
R. E. Leffler	United States Steel Corporation
H. S. Lew	National Bureau of Standards
S. Liapunov	New York University
L. C. Lim	LeMessurier Associates
J. R. Lloyd	Esso Production Research
L. W. Lu	Lehigh University
C. P. Mangelsdorf	University of Pittsburgh
P. W. Marshall	Shell Oil Offshore Construction
J. F. McDermott	United States Steel Corporation
R. M. Meith	Chevron Oil Company
C. D. Miller	Chicago Bridge and Iron Company
S. Morino	Lehigh University
A. Ostapenko	Lehigh University
E. G. Paulet	United States Department of Transportation
T. Pekoz	Cornell University
J. R. Rhodes	University of Strathclyde
N. W. Rimmer	Butler Manufacturing Company
J. B. Scalzi	United States Steel Corporation
C. G. Schilling	United States Steel Corporation
D. R. Sherman	University of Wisconsin - Milwaukee
A. Sieve	Severud Associates
L. Silano	Parsons, Brinckerhoff, Quade and Douglas
J. C. Simonis	Babcock and Wilcox
J. Springfield	Carruthers and Wallace
N. Tebedge	Lehigh University
S. S. Thomaides	Bethlehem Steel Corporation
F. Van der Woude	Lehigh University
E. W. Wright	Carleton University
B. T. Yen	Lehigh University
C. K. Yu	St. Louis University
W. W. Yu	University of Missouri - Rolla
J. A. Yura	University of Texas - Austin

List of Publications

The following papers and reports have been received during the past year. The listing is according to task group and task reporter sequence. Those marked * have been placed in the CRC library.

Task Group 1 - Centrally Loaded Columns

- *Bjorhovde, R., Brozzetti, J., Alpsten, G. A., and Tall, L.
RESIDUAL STRESSES IN THICK WELDED PLATES, Fritz Engineering Laboratory Report No. 337.13, June 1971
- *Bjorhovde, R. and Tall, L.
MAXIMUM COLUMN STRENGTH AND THE MULTIPLE COLUMN CURVE CONCEPT
Fritz Engineering Laboratory Report No. 337.29, September 1971
- *Yu, C. K. and Tall, L.
SIGNIFICANCE AND APPLICATION OF STUB COLUMN TEST RESULTS
Journal, ASCE Struct, Div., Vol. 97, ST7, July 1971
- *Tebedge, N. Alpsten, G., and Tall, L.
MEASUREMENT OF RESIDUAL STRESSES - A STUDY OF METHODS
Fritz Engineering Laboratory Report No. 337.8, February 1971
- *Yu, C. K. and Tall, L.
WELDED AND ROLLED T-1 STEEL COLUMNS - A SUMMARY REPORT,
Fritz Engineering Laboratory Report No. 290.16, June 1960
- *Carpena, A.
DETERMINATION OF THE YIELD POINT FOR COLUMN STRENGTH ANALYSIS
(in French), Construction Métallique, No. 3, 1970
- *Jacquet, J.
COLUMN TESTS AND STATISTICAL ANALYSIS OF THEIR RESULTS
(in French), Construction Métallique, No. 3, 1970

Task Group 3 - Biaxially Loaded Columns

- Chen, W. F. and Santathdaporn, S.
REVIEW OF COLUMN BEHAVIOR UNDER BIAXIAL LOADINGS,
J. of the Structural Division, ASCE, ST12, December 1968,
pp. 2999 - 3021

*Santathadaporn, S. and Chen, W. F.
INTERACTION CURVES FOR SECTIONS UNDER COMBINED BIAXIAL BENDING
AND AXIAL FORCE, WRC Bulletin No. 148, February 1970

*Santathadaporn, S. and Chen, W. F.
ANALYSES OF BIAXIALLY LOADED COLUMNS, Fritz Engineering
Laboratory Report No. 331.12, September 1970

*Santathadaporn, S. and Chen, W. F.
TANGENT STIFFNESS METHOD FOR BIAXIAL BENDING, Fritz
Engineering Laboratory Report No. 331.16, July 1971

Task Group 5 - Classification of Steels for Structures

Task Group 5, Column Research Council
CLASSIFICATION OF STEELS FOR STRUCTURES, AISC Eng. Jnl.,
Vol. 8, No. 3, July 1971, p. 99

Task Group 6 - Test Methods for Compression Members

Tebedge, N., Marek, P., and Tall, L.
ON TESTING METHODS OF HEAVY COLUMNS, Fritz Engineering
Laboratory Report No. 351.4, March 1971

Tebedge, N. and Tall, L.
TEST PROCEDURE OF CENTRALLY LOADED COLUMNS, Fritz Engineering
Laboratory Report No. 351.6, October 1971

Task Group 8 - Dynamic Instability

Herrmann, G. and Krajcinovic, D.
STABILITY OF STRAIGHT BARS SUBJECTED TO REPEATED IMPULSIVE
COMPRESSION, AIAA Jnl., Vol. 6, No. 10, November 1968

Task Group 9 - Curved Compression Members

*Austin, W. J.
IN-PLANE BENDING AND BUCKLING OF ARCHES, Jnl. ASCE Struct.
Div., Vol. 97, No. ST5, May 1971

Task Group 10 - Design of Laterally Unsupported Restrained Beam-Columns

Lim, L. C., Sheninger, E. L., Yoshida, K. and Lu, L. W.
TECHNIQUES FOR TESTING STRUCTURAL SUBASSEMBLAGES WITH BRACED
AND UNBRACED COLUMNS, Fritz Engineering
Laboratory Report No. 329.2, May 1970

Lim, L. C. and Lu, L. W.
 BEHAVIOR OF STRUCTURAL SUBASSEMBLAGES WITH LATERALLY
 UNSUPPORTED COLUMNS, Fritz Engineering Laboratory Report
 No. 329.3, June 1970

Lim, L. C. and Lu, L. W.
 THE STRENGTH AND BEHAVIOR OF LATERALLY UNSUPPORTED COLUMNS,
 Fritz Engineering Laboratory Report No. 329.5, June 1970

Task Group 11 - European Column Studies

*Beer, H. and Schulz, G.
 THEORETICAL BASES OF THE EUROPEAN COLUMN CURVES (in French,
 English translation available), Construction Métallique,
 No. 3, 1970

*Sfintesco, D.
 EXPERIMENTAL BASIS OF THE EUROPEAN COLUMN CURVES (in French)
 Construction Métallique, No. 3, 1970

Task Group 15 - Laterally Unsupported Beams

*Hartmann, A. J.
 EXPERIMENTAL STUDY OF FLEXURAL-TORSIONAL BUCKLING, Jnl. ASCE
 Struct. Div., Vol. 96, No. ST7, July 1970, pp. 1481 - 1493

*Hartmann, A. J.
 INELASTIC FLEXURAL-TORSIONAL BUCKLING, Jnl. ASCE Eng. Mech.
 Div., Vol. 97, No. EM4, August 1971, pp. 1103 - 1119

Task Group 18 - Tubular Members

*Marshall, P. W.
 DESIGN CRITERIA FOR STRUCTURAL STEEL PIPE, Paper presented
 at CRC Annual Technical Session, May 25, 1971

Other Reports

*Johnston, B. G.
 THE CRC GUIDE -A PREVIEW OF THE THIRD EDITION, Meeting Preprint
 1309, ASCE Natl. Water Resources Eng. Mtg., Phoenix,
 January 1970

*Thomas, B. F. and Leigh, J. M.
 THE BEHAVIOUR OF LATERALLY UNSUPPORTED ANGLES, BHP Pty. Ltd.,
 Australia, Publication No. MRL 22/4

Johnston, B. G.
 SPACED STEEL COLUMNS, ASCE Jnl. Struct. Div., Vol 97, No. ST5,
 May 1971

Finance

	Fiscal Year 10/70-9/71		Fiscal Year 10/71-9/72
	Budget	Cash Statement 9/15/70-9/30/71	Budget
<u>BALANCE</u> at Beginning of Period	\$12,800.00	\$15,910.20	\$13,000.00
<u>INCOME</u>			
<u>Contributions</u>			
Aluminum Association	500.00	1,000.00 (1)	500.00
AISC	4,000.00	4,000.00 (2)	4,000.00
AISI	4,000.00	3,750.00	3,500.00
API		1,000.00	1,000.00
CISC	500.00	750.00	750.00
ICBO	100.00		100.00
International Nickel Co.		200.00	
Frank M. Masters	100.00	100.00	100.00
NSF	7,000.00	15,400.00 (3)	13,000.00
SEAONC	25.00		25.00
SEAOSC	100.00		100.00
SESA	100.00		100.00
Total Contributions	16,425.00	26,300.00	23,175.00
<u>Registration Fees</u>	300.00	770.00	800.00
<u>Sale of Publications</u>	200.00		200.00
CRC Secretary		49.72	
J. Wiley & Sons		315.69	
Total Sale		365.41	
<u>Interest</u>	25.00	607.38	25.00
TOTAL INCOME	\$16,950.00	\$28,042.79	\$24,200.00
<u>EXPENDITURES</u>			
<u>Technical Services</u> (Headquarters)			
Director	1,500.00	1,500.00	1,500.00
Sec. & Steno Services		5,957.54	
Material & Supply	7,000.00	1,655.77	7,000.00
Travel		133.12	
Total	8,500.00	9,246.43	8,500.00
<u>Research</u>			
TG 1 - Lehigh University			
TG 4 - New York University		1,000.00 (4)	1,000.00
TG 15 - Meeting Expense	1,000.00	1,000.00	
TG 17 - Univ. Of Missouri, Columbia			300.00
<u>Guide</u>			1,200.00
Contract on revision	3,600.00	3,600.00	4,800.00
Travel and Expense	1,200.00	2,613.95	4,700.00
Purchase (for distr. & sale)		15.64	
<u>Purchase of Technical Papers</u>	250.00	600.00	500.00
<u>United Engineering Trustees</u>	100.00	100.00	100.00
<u>Annual Meeting and Proceedings</u>			
Expenses and services	4,000.00	3,247.70	3,500.00
Travel	3,000.00	2,101.86	2,500.00
Other		8,523.70 (5)	
<u>Travel</u>		226.91	500.00
<u>Contingencies</u>	400.00	26.00 (6)	400.00
TOTAL EXPENDITURES	\$22,050.00	\$32,302.19	\$28,000.00
<u>BALANCE</u> at End of Period	\$ 7,700.00	\$11,650.80	\$ 9,200.00

F I N A N C E (C o n t i n u e d)DEPOSITORIES (As of 30Sep71)

<u>Engineering Foundation</u>	\$ 4,074.22
<u>Lehigh University Account</u>	155.97 (Deficit)
<u>Unexpended Balance on NSF Contracts</u>	
CRC Guide	4,107.38
1971 Annual Meeting	<u>3,625.17</u>
 TOTAL DEPOSITS	 \$11,650.80

EXPLANATORY NOTES

- (1) Including \$500 contribution for 1971-72
- (2) Including \$1,500 for CRC Guide Support
- (3) \$9,600 grant for CRC Guide preparations; \$5,800 grant for 1971 Annual Meeting. Funds received upon reimbursement of bills by Lehigh University
- (4) Budgeted for 1971-72 but paid in 1970-71
- (5) Partial expenditures for the 1968 & 69 Annual Meeting, not included in previous statements
- (6) Task Group 18 meeting expense

Register

OFFICERS

Chairman	T. V. Galambos
Vice Chairman	G. Winter
Secretary	F. Van der Woude *

EXECUTIVE COMMITTEE

T. V. Galambos (73)
 G. Winter (73)
 L. S. Beedle (Director)
 J. W. Clark (72)
 T. Dembie (74)
 J. L. Durkee (73)**
 E. H. Gaylord (73)*
 J. A. Gilligan (74)
 T. R. Higgins (Technical Consultant)
 I. M. Hooper (73)
 J. S. B. Iffland (73)
 L. K. Irwin (74)
 B. G. Johnston (72)
 W. A. Milek, Jr. (73)
 C. F. Scheffey (72)

* Past Chairman

** Past Vice Chairman

STANDING & AD HOC COMMITTEES

A. Committee on the Guide to Design Criteria for Metal Compression Members (Appointments expire 1973)

E. H. Gaylord, Chmn.	G. F. Fox	B. G. Johnston
A. M. Amirikian	T. V. Galambos	R. G. Kline
W. J. Austin	J. A. Gilligan	C. F. Scheffey
L. S. Beedle	G. Haaier	F. D. Sears
K. P. Buchert	R. L. Haenel	D. Sfintesco
J. W. Clark	T. R. Higgins	J. Springfield
D. A. DaDeppo	J. S. Iffland	L. Tall
J. L. Durkee	S. H. Iyengar	G. Winter
S. J. Errera	A. L. Johnson	

B. Committee on Finance

L. S. Beedle, Chairman
 T. V. Galambos
 F. Van der Woude*

C. Ad Hoc Committee on Research

T. V. Galambos, Chairman
 Members of Executive Committee

* Present Secretary: G. W. Schulz

TASK GROUPS

Task Group 1 - Centrally Loaded Columns

J. A. Gilligan, Chairman *	J. E. Goldberg	E. G. Paulet
R. R. Graham, Vice Chairman	D. H. Hall	L. Plofker
L. S. Beedle	R. L. Ketter	L. D. Sandvig
C. E. Cutts	A. F. Kirstein	L. Tall
M. P. Gaus	W. A. Milek, Jr.	

Task Group 1 is concerned with the strength of centrally loaded columns as influenced by geometrical properties of the column cross section, mechanical properties of the material in the column and variables associated with the manufacture and fabrication of columns.

Task Group 3 - Ultimate Strength of Columns With Biaxially Eccentric Load

J. Springfield, Chairman	E. H. Gaylord*	B. C. Ringo
T. Dembie	L. W. Lu	G. Ruplev
J. S. Ellis		

This task group is concerned with investigating the behavior of columns subjected to biaxial bending, with a view of developing rational design procedures based on the ultimate strength of such members.

Task Group 4 - Frame Stability and Effective Column Length

J. S. B. Iffland, Chairman*	M. S. Gregory	L. W. Lu
P. F. Adams	O. Halasz	W. A. Milek, Jr.
C. Birnstiel	T. R. Higgins	C. K. Wang
W. E. Edwards	I. J. Hooper	
E. H. Gaylord	B. G. Johnston	

The purpose of this task group is to investigate the stability of building frames, including effective column length aspects. It will work in close contact with Task Groups 10 and 15.

Task Group 6 - Test Methods for Compression Members

L. Tall, Chairman	E. W. Gradt	B. G. Johnston
C. K. Yu, Vice Chairman	R. A. Hechtman	B. M. McNamee
L. S. Beedle	T. R. Higgins	H. H. Tung
J. W. Clark	L. K. Irwin*	

This task group is concerned with the development of technical memoranda on experimental methods and techniques of testing structural members subject to buckling, including the analysis of the data of the test. It is also the purpose of the group to organize and conduct technical sessions and symposia on test methods to facilitate exchange of information on new testing procedures.

*Executive Committee Contact Member

Task Group 7 - Tapered Members (Joint Task Group with WRC)

A. Amirikian, Chairman	R. L. Ketter	N. R. Rimmer
C. F. Larson, Secretary	K. H. Koopman	A. A. Toprac
J. H. Adams	G. C. Lee	I. M. Viest
D. J. Butler	L. W. Lu	
T. R. Higgins*	W. A. Milek, Jr.	

This task group, a joint task group with Welding Research Council, is concerned with research leading to the development of design procedures for tapered structural members and frames made of such members.

Task Group 8 - Dynamic Instability

D. A. DaDeppo, Chairman	I. K. McIvor
B. G. Johnston*	J. C. Simonis
D. Krajcinovic	

The goal of the work of this task group is to make design recommendations regarding the load carrying capacity of columns and other compression members subjected to dynamic loading. To this end, the available information in field will be correlated and the areas in which further research effort is required will be identified.

Task Group 9 - Curved Compression Members

W. J. Austin, Chairman	N. C. Lind	M. Ojalvo
S. O. Asplund	J. A. Mandel	A. Stiev
J. Chinn	N. G. Marks	L. G. Silano
J. W. Clark*	E. F. Masur	G. A. Wempner

This task group is concerned with the stability of curved compression members, such as arches, loaded in the plane of curvature. Both in-plane and lateral buckling are to be considered. The task group aims at the development of information to be used in a new chapter of the Guide to Design Criteria for Metal Compression Members.

Task Group 10 - Design of Laterally Unsupported Restrained Beam-Columns

T. V. Galambos, Chairman*	G. C. Lee	W. A. Milek, Jr.
J. A. Gilligan	L. W. Lu	M. Ojalvo

This task group is concerned with the study of design methods for wide-flange beam-columns subjected to strong axis bending and unbraced against out-of-plane deformations. The study consists of experimental and analytical investigations of the behavior of beam-and-column assemblages where the columns are laterally unrestrained. The final purpose is the development of improved design rules for such members.

Task Group 11 - European Column Studies

D. Sfintesco, Chairman	C. A. Cornell	E. O. Pfrang
W. A. Milek, Jr. Vice Chairman*	M. P. Gaus	J. Strating
G. A. Alpsten	R. K. McFalls	L. Tall
L. S. Beedle	B. M. McNamee	I. M. Viest
A. Carpena	P. Marek	C. K. Yu

The purpose of this task group is to examine the strength of centrally loaded steel columns with particular reference to a statistical approach to tests and interpretation of data. Through collaboration with Subcommittee 8 of the European Convention of Constructional Steel work, the task group will provide guidance to experimental and theoretical studies in the United States of the heavier European rolled shapes.

Task Group 12 - Mechanical Properties of Steel in Inelastic Range

G. F. Fox, Chairman	A. F. Kirstein	C. F. Scheffey*
J. J. Healey	L. W. Lu	W. J. Wilkes

The purpose of the task group is to obtain data on the mechanical properties of steel in the inelastic range of particular importance to stability solutions. Among other things this would include determination of the average value and variation of the following: yield stress level, strain hardening modulus, magnitude of strain at initial strain hardening, and, for materials without a well defined yield point, yield strength, tangent modulus and secant modulus.

Task Group 13 - Thin-Walled Metal Construction

S. J. Errera, Chairman	J. A. Gilligan	T. Pekoz
J. W. Clark	A. L. Johnson	G. Winter*
E. R. Estes, Jr.	A. Ostapenko	W. W. Yu

The purpose of this task group is to digest the literature on thin-walled metal construction, as it relates to stability, and to draft a chapter for the third edition of the CRC Guide. Materials of interest include carbon steels, alloy steels, stainless steels and aluminum alloys. The effects of various manufacturing and fabrication processes shall be considered.

Task Group 14 - Horizontally Curved Girders

C. F. Scheffey, Chairman*	C. G. Culver	M. Ojalvo
R. Behling	P. Marek	S. Shore
H. R. Brannon	W. A. Milek, Jr.	W. M. Thatcher

The purpose of this task group is to explore the stability problems which occur in horizontally curved girders, both during erection and in the completed structure, the effects of rolling and fabrication practice on these problems, and criteria for adequate bracing.

Task Group 15 - Laterally Unsupported Beams

R. L. Haenel, Chairman	T. V. Galambos*	A. J. Hartmann
P. F. Adams	W. Hansell	J. A. Yura

The purpose of this task group is to study the stability of laterally unsupported beams and the bracing requirements for such beams in both the elastic and inelastic ranges with emphasis on beams in framed structures. The research should lead to a design procedure for such members.

Task Group 16 - Plate and Box Girders

F. D. Sears, Chairman	K. L. Heilman	A. Ostapenko
K. Basler	H. S. Lew	B. T. Yen
P. B. Cooper	C. Massonnet	
J. L. Durkee*		

This task group is concerned with the stability and strength of plate girders. A considerable amount of work on the behavior and load carrying capacity of plate girders is underway in this and other countries. The purposes of the task group are to facilitate exchange of information among these investigators, to encourage preparation of reports relevant to design specifications, and to assist in revising the chapter on plate girders in the CRC Guide.

Task Group 17 - Stability of Shell-Like Structures

K. P. Buchert, Chairman	T. V. Galambos	E. P. Popov
J. H. Adams	A. Kalnins	C. F. Scheffey
L. O. Bass	D. Krajcinovic	D. R. Sherman
A. Chajes	C. Libove	J. C. Simonis
J. W. Clark*	C. D. Miller	D. T. Wright
J. O. Crooker		

The purpose of this task group is to prepare a chapter for the CRC Guide, summarizing design information on the stability of civil engineering shell-type structures.

Task Group 18 - Tubular Members

A. L. Johnson, Chairman	S. C. Fan	P. W. Marshall
M. P. Bernstein	D. W. Fowler	R. M. Meith
L. A. Boston	R. R. Graham	C. D. Miller
A. Chajes	J. R. Lloyd	R. L. Rolf
J. L. Durkee*	J. N. Macadam	D. R. Sherman
N. W. Edwards		

The purpose of this task group is to prepare a chapter for the Guide. This chapter will summarize design information on cylindrical tubes and shells.

Task Group 19 - Stiffened Plate Structures

R. G. Kline, Chairman	R. Glasfeld	M. L. Sharp
P. J. Fang	H. G. Harris	
J. A. Gilligan*	A. Ostapenko	

The purpose of this task group is to prepare material for the Guide concerning stiffened plate structures.

Task Group 20 - Composite Members

S. H. Iyengar, Chairman
L. S. Beedle*

TASK REPORTERS

Task Reporter 11 - Stability of Aluminum Structural Members

J. W. Clark, Aluminum Company of America

Task Reporter 13 - Local Inelastic Buckling

L. W. Lu, Lehigh University

Task Reporter 14 - Fire Effects on Structural Stability

L. S. Seigel, U. S. Steel Corporation

PARTICIPATING ORGANIZATIONS

<u>Organization</u>	<u>Representatives</u>	<u>Officers</u>
Aluminum Association	J. W. Clark C. M. Tyler, Jr.	Paul V. Mara, Technical Director
American Association of State Highway Officials	H. W. Derthick	A. E. Johnson, Executive Secretary
American Institute of Architects	A. G. Lorimer	J. R. Dowlings, Dir. Codes & Reg.
American Institute of Consulting Engineers	E. R. Hardesty F. M. Masters E. K. Timby	E. L. Wemple, Secretary
American Institute of Steel Construction	T. R. Higgins J. L. Durkee	W. A. Milek, Jr., Dir. of Engr. & Res.
American Iron and Steel Institute	W. G. Kirkland A. L. Johnson	L. A. Baron
American Petroleum Institute	L. A. Boston J. E. Ubben	J. H. Sybert
American Society of Civil Engineers	S. C. Hollister B. G. Johnston T. C. Kavanagh	W. H. Wisely, Executive Director
American Society of Mechanical Engineers	R. B. Allnutt M. B. Bernstein J. C. Simonis	O. B. Schier, II, Executive Director
American Water Works Association	E. F. Johnson	E. F. Johnson, Executive Secretary
Association of American Railroads	L. S. Beedle F. P. Drew	E. W. Hodgkins, Executive Secretary
Boston Society of Civil Engineers	J. M. Biggs	J. F. Brittain, President C. E. Baird, Secretary
Federal Highway Administration	W. J. Wilkes C. F. Scheffey	F. C. Turner Administrator
Canadian Institute of Steel Construction	T. Dembie H. A. Krentz	R. G. Johnson, President
Corps of Engineers U. S. Army	W. A. Heitmann	Lt. Gen. F. J. Clarke, Chief
Earthquake Engineering Research Institute	H. Degenkolb R. D. Hanson T. R. Higgins	C. M. Duke, President
Engineering Foundation		J. A. Zecca, Secretary
Engineering Institute of Canada	J. S. Ellis R. F. Hooley D. T. Wright	P. Bournival, General Manager

<u>Organization</u>	<u>Representatives</u>	<u>Officers</u>
European Convention of Constructional Steelworks		D. Sfintesco, Technical Sec. General E. B. Evers, Administrative Sec. Gen
General Services Administration	B. C. Berube	A. F. Sampson, Commissioner, Public Buildings Service
International Conference of Building Officials	J. E. Bihr	J. E. Bihr, Technical Director
International Nickel Company, Inc.	A. C. Kuentz	
Langley Research Center National Aeronautics & Space Administration	M. Stein	E. M. Cortright, Director
Metal Building Manufacturers Association	N. W. Rimmer F. A. Petersen	F. A. Petersen, General Manager
National Bureau of Standards	L. K. Irwin E. O. Pfrang A. F. Kirstein	L. M. Branscomb, Director
National Research Council	R. D. Edgerton	J. S. Coleman, Executive Officer Admiral L. R. Daspit
Naval Facilities Engineering Command, U. S. Navy	A. Amirikian W. Shaw	Rear Adm. W. N. Enger (CEC), USN
Society for Experimental Stress Analysis	C. S. Barton N. J. Hoff N. M. Newmark	B. E. Rossi, Executive Secretary
Steel Joist Institute	J. D. Johnson	R. B. Woodward, Managing Director
Structural Engineers Association of Northern California	G. V. Jacobs L. A. Napper H. P. Weldon	H. R. Hammill, President William W. Giles, Jr., Executive Secretary
Structural Engineers Association of Southern California	R. W. Binder J. O. Robb G. D. Lehmer	Don Wiltse, Executive Secretary
Welding Research Council	R. L. Ketter L. Grover W. A. Milek	K. H. Koopman, Director
Western Society of Engineers	J. F. Parmer	F. R. Bruce, Executive Secretary

REPRESENTATIVES OF PARTICIPATING ORGANIZATIONS

<u>Representative</u>	<u>Organization</u>
R. B. Allnutt	American Society of Mechanical Engineers
A. Amirikian	Naval Facilities Engineering Command, U. S. Navy
C. S. Barton	Society for Experimental Stress Analysis
L. S. Beedle	Association of American Railroads
B. C. Berube	General Services
M. B. Bernstein	American Society of Mechanical Engineers
J. M. Biggs	Boston Society of Civil Engineers
J. E. Bihr	International Conference of Building Officials
R. W. Binder	Structural Engineers Association of Southern California
L. A. Boston	American Petroleum Institute
J. W. Clark	Aluminum Association
H. Degenkilb	Earthquake Engineering Research Institute
T. Dembie	Canadian Institute of Steel Construction
H. Derthick	American Association of State Highway Officials
F. P. Drew	Association of American Railroads
J. L. Durkee	American Institute of Steel Construction
R. C. Edgerton	National Research Council
J. S. Ellis	Engineering Institute of Canada
L. Grover	Welding Research Council
R. D. Hanson	Earthquake Engineering Research Institute
E. R. Hardesty	American Institute of Consulting Engineers
W. A. Heitmann	Corps of Engineers, U. S. Army
T. R. Higgins	American Institute of Steel Construction
N. J. Hoff	Society for Experimental Stress Analysis
S. C. Hollester	American Society of Civil Engineers
R. F. Hooley	Engineering Institute of Canada
L. K. Irwin	National Bureau of Standards
G. V. Jacobs	Structural Engineers Association of Northern California
A. L. Johnson	American Iron and Steel Institute
E. F. Johnson	American Water Works Association
J. D. Johnson	Steel Joist Institute
B. G. Johnston	American Society of Civil Engineers
T. C. Kavanagh	American Society of Civil Engineers
R. L. Ketter	Welding Research Council
W. G. Kirkland	American Iron and Steel Institute
A. F. Kirstein	National Bureau of Standards
H. A. Krentz	Canadian Institute of Steel Construction
A. C. Kuentz	International Nickel Company, Inc.
G. D. Lehmer	Structural Engineers Association of Southern California
A. G. Lorimer	American Institute of Architects

RepresentativeOrganization

F. M. Masters	American Institute of Consulting Engineers
W. A. Milek	Welding Research Council
L. A. Napper	Structural Engineers Association of Northern California
N. M. Newmark	Society for Experimental Stress Analysis
J. F. Parmer	Western Society of Engineers
 J. P. Petersen	 Langley Research Center, National Aeronautics and Space Administration
E. O. Pfrang	National Bureau of Standards
N. W. Rimmer	Metal Building Manufacturers Association
J. O. Robb	Structural Engineers Association of Southern California
C. F. Scheffey	Federal Highway Administration
 W. Shaw	 Naval Facilities Engineering Command, U. S. Navy
J. C. Simonis	American Society of Mechanical Engineers
M. Stein	Langley Research Center, National Aeronautics and Space Administration
E. K. Timby	American Institute of Consulting Engineers
C. M. Tyler	Aluminum Association
 J. E. Ubben	 American Petroleum Institute
H. P. Weldon	Structural Engineers Association of Northern California
W. J. Wilkes	Federal Highway Administration
D. T. Wright	Engineering Institute of Canada

MEMBERS-AT-LARGE

P. F. Adams	W. H. Jameson	A. I. Westrich
R. J. Alvarez	O. C. Julian	G. Winter
W. J. Austin		R. N. Wright
	F. Khan	W. Wright
R. M. Barnoff	A. C. Kuentz	
R. H. Batterman		B. T. Yen
C. Birnstiel	G. C. Lee	J. A. Yura
H. H. Bleich	W. J. LeMessurier	
J. G. Bouwkamp	F. J. Lin	
K. P. Buchert	L. W. Lu	
A. Chajes	P. Marek	
W. F. Chen	E. F. Masur	
J. W. Clark	G. F. McDonough, Jr.	
P. B. Cooper	J. McDonough	
C. G. Culver	I. K. McIvor	
C. E. Cutts	B. M. McNamee	
	J. Michalos	
G. C. Driscoll, Jr.	T. Murray	
J. E. Duberg		
J. L. Durkee	P. K. Neogi	
A. L. Elliott	M. Ojalvo	
E. L. Erickson	A. Ostapenko	
S. J. Errera		
E. R. Estes, Jr.	F. J. Palmer	
	E. G. Paulet	
G. P. Fisher	S. U. Pillai	
G. F. Fox	L. Plofker	
T. V. Galambos	B. C. Ringo	
C. N. Gaylord	L. E. Robertson	
E. H. Gaylord	A. R. Robinson	
J. M. Gere	J. P. Rutigliano	
J. A. Gilligan		
J. E. Goldberg	C. F. Scheffey	
E. W. Gradt	C. G. Schilling	
R. R. Graham	W. C. Schnobrich	
	E. E. Sechler	
G. Haaiker	S. Shore	
R. L. Haenel	J. Springfield	
D. H. Hall	H. K. Stephenson	
A. J. Hartmann		
B. J. Hartz	L. Tall	
R. A. Hectman	W. Teng	
G. Herrmann	S. S. Thomaides	
E. C. Holt	A. A. Toprac	
I. Hooper	N. S. Trahair	
T. Huang		
C. L. Hulsbos	N. Van Eenam	
	I. M. Viest	
J. S. B. Iffland		

CORRESPONDING MEMBERS

G. Augusti	Italy
H. Beer	Austria
F. S. Braga	Brazil
R. M. Clark	South Africa
G. B. Godfrey	England
M. S. Gregory	Tasmania
O. Halasz	Hungary
T. Hayashi	Japan
J. Heyman	England
N. S. Koo	People's Repub of China
M. G. Lay	Australia
J. A. P. Laurie	South Africa
M. A. Leclair	Nicaragua
G. D. Malo	Panama
C. Massonnet	Belgium
G. Nassar	Egypt
H. Nylander	Sweden
A. Selberg	Norway
D. Sfintesco	France
F. S. Shaw	Australia
R. Solis	Guatemala
K. Thomsen	Denmark
B. Thürlimann	Switzerland
U. Vogel	West Germany
Secretary, New Zealand Institute of Engineers	New Zealand

ADDRESSES OF PARTICIPANTS*

- ADAMS, J. H., Chief Engineer, Pittsburgh DesMoines Steel Co., Neville Island, Pittsburgh, Pennsylvania 15225
- ADAMS, Dr. Peter F., Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada
- ALLNUTT, Dr. R. B., Structures Evaluation Branch, Naval Ship Research Development Center, Navy Department, Washington, D.C. 20007
- ALPSTEN, Dr. Goran A., Swedish Institute of Steel Construction, Drottning Kristinas Vag 48, S-114 28, Stockholm, Sweden
- ALVAREZ, Prof. R. J., Engineering Science and Mechanics, Hofstra University, Hempstead, Long Island, New York, New York 11550
- AMIRIKIAN, Dr. A., Chief Engineering Advisor, Naval Facilities Engineering Command, Department of the Navy, Washington, D.C. 20390
- ANSELMINI, Ludwig, Application Engineer, The International Nickel Co., Inc., 67 Wall Street, New York, New York
- ASPLUND, Prof. Dr. S. O., Berggrensgatan 12, 412 73 Göteborg, Sweden
- AUGUSTI, Prof. Guiliano, Ist. Scienza delle Costruzioni, Universita (Fac. Ingegneria), 80125 Napoli, Italy
- AUSTIN, Prof. W. J., Department of Civil Engineering, Rice University, P.O. Box 1892, Houston Texas 77001
- BAIRD, C.E., Secretary, Boston Society of Civil Engineers, 47 Winter Street, Boston, Massachusetts 02108
- BAKER, Prof. J. F., Department of Engineering, Cambridge University, Cambridge, England
- BARNOFF, Prof. Robert M., Department of Civil Engineering, Pennsylvania State University, 212 Sackett Building, Univ. Park, Penna. 16802
- BARRON, Leslie A., Vice-President, Engineering, AISI, 633 Third Avenue, New York, New York 10017
- BARTLESMEYER, R.B., Bureau of Public Roads, Federal Highway Administration, Washington, D.C. 20591
- BARTON, Dr. C. S., Department of Civil Engineering Science, Brigham Young University, Provo, Utah 84601
- BASLER, Dr. K., Forchstrasse 84, 8008 Zurich, Switzerland
- BASS, Louis O., 1705 West 5th, Stillwater, Oklahoma 74074
- BATTERMAN, Richard H., Research Engineer, Alcoa Research Laboratories, Aluminum Company of America, New Kensington, Pennsylvania 15068

* Please see the Appendix for additions made after October 1, 1971.

- BEAUFAIT, Prof. F., S.T.D. Newsletter Editor, Box 1533, Station B,
Vanderbilt University, Nashville, Tennessee
- BEEDLE, Dr. L. S., Director, Fritz Engineering Laboratory, Lehigh University,
Bethlehem, Pennsylvania 18015 (102 Cedar Rd., Hellertown, Pa. 18055)
- BEER, Prof. Hermann O., Prof. für Stahlbau, Holzbau und Flächentragwerke,
Technische Hochschule Graz, Graz, Rechbauerstrasse 12 A8010, Austria
- BEHLING, R., State Highway Department, Salt Lake City, Utah
- BERNSTEIN, Martin P., Foster Wheeler Corporation, 110 South Orange Avenue,
Livingstone, New Jersey 07039
- BERUBE, Bertrand C., Public Buildings Service, General Services Administration
18th and F. Streets, N.W., Washington, D.C. 20405
- BIGGS, Prof. John M., Massachusetts Institute of Technology, 99 Massachusetts
Avenue, Cambridge, Massachusetts 02139
- BIHR, James E., Technical Director, International Conference of Building
Officials, 50 S. Los Robles, Pasadena, California 91101
- BINDER, R.W., 9539 Sawyer Street, Los Angeles, California 90035
- BIRNSTIEL, Prof., Charles, Department of Civil Engineering, New York
University, University Heights, New York, New York 10453
- BLEICH, Prof. H. H., Department of Civil Engineering & Engineering Mechanics,
Columbia University, New York, New York 10027
- BLETZACHER, Prof. R. W., Building Research Lab., Ohio State University,
Room 417 Hitchcock Hall, 2070 Neil Avenue, Columbus, Ohio 43210
- BODHE, J.G., Chairman, Civil Engineering Division, The Institution of
Engineers, 8, Gokhale Road, Calcutta 20, India
- BOSTON, L. A., Cities Service Oil Company, Box 300, Tulsa, Oklahoma 74102
- BOURNIVAL, P., General Manager, Engineering Institute of Canada, 2050
Mansfield Street, Montreal, Quebec, Canada
- BOUWKAMP, Prof. J. G., Department of Civil Engineering, University of
California, Berkeley, California 94720
- BRAGA, F. Saturnino, Director General, Department Nacional de Estradas de
Rodegem, Rio de Janeiro, D. F., Brazil, Argentina
- BRANNON, H. R., Esso Production Research Company, P.O. Box 2189, Houston,
Texas
- BRANSCOMB, Dr. Lewis, Director, National Bureau of Standards, Washington,
D.C. 20234
- BRITTAIN, James F., President, Boston Society of Civil Engineers, 47 Winter
Street, 8th Floor, Boston, Massachusetts 02108

- BRUCE, Fred R., Executive Secretary, Western Society of Engineers, 314 South Federal Street, Chicago, Illinois 60604
- BRYANS, R.F., American Institute of Aeronautics & Astronautics, 1290 6th Avenue, New York, New York 10019
- BUCHERT, Prof. K. P., Department of Civil Engineering, University of Missouri, Columbia, Missouri 65201
- BUTLER, Prof. D. J., Department of Civil Engineering & Engineering Mechanics, Columbia University, New York, New York 10027
- CARLSON, Roy F., Director, American Petroleum Institute, 300 Corrigan Tower Building, Dallas, Texas 75201
- CARPENA, Dr. A., c/o S.A.E., Via G. Fara 26, Milano, Italy
- CARPENTER, Lauren D., Skidmore, Owings & Merrill, 30 West Monroe Street, Chicago, Illinois 60607
- CASSIDY, Lt. Gen. William F., Chief of Engineers, Department of the Army, Washington, D.C. 20315
- CHAJES, Alexander, Assoc. Prof., University of Massachusetts, Department of Civil Engineering, Amherst, Massachusetts 01002
- CHEN, Dr. W. F., Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015
- CHENG, Dr. F. Y., Assoc. Prof., Department of Civil Engineering, University of Missouri, Rolla, Missouri
- CHINN, Prof. James, Civil Engineering Department, University of Colorado, Boulder, Colorado 80302
- CLARK, Dr. John W., Chief, Engineering Design Division, Alcoa Research Laboratories, P.O. Box 772, New Kensington, Pennsylvania 15068
- CLARK, R. McHaffie, Director, South African Institute of Steel Construction, Ltd., P.O. Box 1338, Johannesburg, South Africa
- COLEMAN, John S., Executive Officer, National Academy of Sciences, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418
- COLLIN, A. L., Senior Development Engr., Kaiser Steel Corp., 300 Lakeside Drive, Oakland, California 94612
- COOPER, Prof. P. B., Department of Civil Engineering, Seaton Hall, Kansas State University, Manhattan, Kansas 66502
- CORNELL, Prof. C. Allin, Department of Civil Engineering 1-263, Massachusetts Institute of Technology, Cambridge, Massachusetts 02139
- CROOKER, James O., Jr., Design Engineer, Butler Manufacturing Company, 7400 East 13th Street, Kansas City, Missouri

- CULVER, Prof. Charles G., Carnegie-Mellon University, 5000 Forbes Avenue,
Pittsburgh, Pennsylvania 15213
- CUTTS, Prof. Charles E., Head, Department of Civil Engineering, Michigan
State University, East Lansing, Michigan 48823
- DA DEPPO, Prof. D. A., Department of Civil Engineering, University of
Arizona, Tucson, Arizona 85721
- DASPIT, R. Adm. L. R., Executive Secretary, Division of Engineering, National
Research Council, 2101 Constitution Ave., N.W., Washington, D.C. 20418
- DAVIS, C.S., University of Missouri, Rolla, Missouri
- DEGENKOLB, Henry J., Degenkolb & Associates, Consulting Engineers, 350
Sansome Street, San Francisco, California 94104
- DEMBIE, Thomas, Chief Engineer, Dominion Bridge Company, Ltd., Box 310 -
Terminal "A", Toronto, Ontario, Canada
- DERTHICK, H. W., Assistant Bridge Engineer, Tennessee Department of Highways,
1010 Highway Building, Nashville, Tennessee 37219
- DE SAINT MALO, Guillermo, Decsno, Facultad de Ingenieria, Universidad de
Panama, Panama City, Republic of Panama
- DONLAN, C. J., Acting Director, Langley Research Center, National
Aeronautics & Space Administration, Hampton, Virginia 23365
- DREW, Freeman P., Research Engineer, Structures, Association of American
Railroads, 3140 South Federal Street, Chicago, Illinois 60616
- DRISCOLL, Prof. G. C., Jr., Fritz Engineering Laboratory, Lehigh University,
Bethlehem, Pennsylvania 18015
- DUBERG, Dr. J. E., 4 Museum Drive, Newport News, Virginia 23601
- DUKE, Prof. D. Martin, President, EERI, College of Engineering, University
of California, Los Angeles, California
- DURKEE, Jackson L., Chief Engineer - Bridges, Fabricated Steel Construction,
Bethlehem Steel Corp., Bethlehem, Pennsylvania 18016
- EDGERTON, Roy D., Director, Technical Activities, Highway Research Council,
2101 Constitution Avenue, N.W., Washington, D.C. 20418
- EDWARDS, W. E., Engineer - Buildings, Bethlehem Steel Company, Room B259,
North Building, Bethlehem, Pennsylvania 18015
- ELLIOTT, Arthur L., Bridge Engineer, California Division of Highways,
P.O. Box 1499, Sacramento, California 95807
- ELLIS, Dr. J. S., Department of Civil Engineering, Royal Military College
of Canada, Kingston, Ontario, Canada

- ENGER, W. M., Rear Admiral, Commander, Naval Facilities Engineering Command,
Department of the Navy, Washington, D.C. 20390
- ERICKSON, E. L., Consulting Engineer, 501 Dumbarton Street, Shreveport,
Louisiana 71106
- ERRERA, Dr. S. J., Homer Research Laboratory, Bethlehem Steel Corporation,
Bethlehem, Pennsylvania 18016
- ESTES, Edward R., Jr., Engineering Manager - Special Development, Republic
Steel Corporation, Manufacturing Division, 1315 Albert Street,
Youngstown, Ohio 44505
- EVERS, E. B., Administrative Secretary General, E.C.C.S., Weena 700,
Rotterdam, The Netherlands
- FANG, Dr. P. J., Department of Civil & Environmental Engineering, University
of Rhode Island, Kingston, Rhode Island 02881
- FARWELL, Charles R., Jr., Senior Engineering, McDonnell-Douglas Aircraft
Corp., Lambert-St. Louis Municipal Airport, Box 516, St. Louis,
Missouri 63166
- FISHER, Dr. Gordon P., Head, Department of Environmental Systems Engineering,
Cornell University, Ithaca, New York 14850
- FOWLER, Asst. Prof. David W., Department of Architectural Engineering,
University of Texas, Austin, Texas 78712
- FOX, G. F., Howard, Needles, Tammen & Bergendoff, 1345 Avenue of the
Americas, New York, New York 10019
- GALAMBOS, Prof. T. V., Department of Civil & Environmental Engineering, BOX 1130
Washington University, St. Louis, Missouri 63130
- GAUS, Dr. M. P., Assistant Program Director, National Science Foundation,
1951 Constitution Avenue, Washington, D.C. 20550
- GAYLORD, Prof. Charles N., Chairman, Department of Civil Engineering,
Thornton Hall, Univ. of Virginia, Charlottesville, Virginia 22901
- GAYLORD, Prof. E. H., Department of Civil Engineering, 2129 Civil Engineering
Building, University of Illinois, Urbana, Illinois 61801
- GERE, Prof. J. M., Civil Engineering Department, Stanford University,
Stanford, California 94305
- GHOSH, Shri D. K., Officiating Secretary, The Institution of Engineers
(India), 8 Gokhale Road, Calcutta 20, India
- GILES, William W., Executive Secretary, Structural Engineers Association of
Northern California, 171 Second St., San Francisco, California 94105
- GILLAN, Gerald K., Civil Engineer, Modjeski & Masters, P.O. Box 2345,
Harrisburg, Pennsylvania 17105

- GILLIGAN, John A., United States Steel Corporation, 600 Grant Street,
Pittsburgh, Pennsylvania 15230
- GLASFELD, Dr. Rolf, General Dynamics Corporation, Quincy Shipbuilding
Division, Department 867, 97 Howard Street, Quincy, Mass. 02169
- GODFREY, G. B., Constructional Steel Research and Development Organization,
Albany House, Petty France, London, S.W. 1, England
- GODFREY, K. A., Editor, A.S.C.E., 345 East 47th Street, New York, New
York 10017
- GOLDBERG, Prof. J. E., Structural Engineering, Purdue University,
Lafayette, Indiana 47907
- GRADT, E. W., American Iron & Steel Institute, 633 Third Avenue, New York,
New York 10017
- GRAHAM, R. R., United States Steel Corporation, 600 Grant Street, Room 1779,
Pittsburgh, Pennsylvania 15230
- GREGORY, Dr. M. S., Civil Engineering Department, The University of
Tasmania, Box 252C, G.P.O., Hobart, Tasmania 7001, Australia
- GROVER, LaMotte, Powder Glen, R.D. #2, Wapwallopen, Pennsylvania 18660
Box 12
- HAAIJER, Dr. Geerhard, Division Chief, Applied Research Laboratory, United
States Steel Corporation, Monroeville, Pennsylvania 15146
- HAENEL, Robert L., P.E. Associate, Larsen & Ludwig, 1102 Century Building,
Pittsburgh, Pennsylvania 15222
- HALASZ, Dr. Otto, Dean, School of Civil Engineering, Technical University,
Budapest XI, Hungary
- HALL, D. H., Associate Engineer, Bethlehem Steel, Homer Research Laboratory,
Bethlehem, Pennsylvania 18015
- HAMMILL, H. Robert, 657 Howard Street, San Francisco, California 94105
- HANSON, Prof. Robert D., Department of Civil Engineering, University of
Michigan, Ann Arbor, Michigan 48104
- HARRIS, Dr. Harry G., Group 461, Plant 35, Grumman Aerospace Corporation,
Bethpage, New York 11714
- HARRIS, Prof. Robert B., Department of Civil Engineering, University of
Michigan, Ann Arbor, Michigan 48104
- HARTMANN, Prof. A. J., Department of Civil Engineering, Marquette University,
Milwaukee, Wisconsin 53233

- HARTZ, Prof. B. J., Department of Civil Engineering, University of Washington, Seattle, Washington 98105
- HAYASHI, Prof. Tsuyoshi, Chairman, Column Research Committee of Japan, University of Tokyo, Hongo 7-3-1, Bunkyo-ku, Tokyo, Japan
- HEALY, Dr. John J., Ammann & Whitney, 111 Eighth Avenue, New York, New York 10011
- HECHTMAN, Dr. Robert A., 821 Clinton Place, McLean, Virginia 22101
- HEGNER, C. F., Commissioner, Public Buildings Service, General Services Administration, 18th & F Streets, N.W., Washington, D.C. 20405
- HEILMAN, K. L., Pennsylvania Department of Transportation, P.O. Box 2926, Harrisburg, Pennsylvania 17120
- HEITMANN, William A., Chief, Structural Section, Directorate of Military Constr., Office of Chief of Engineers, Department of the Army, Washington, D.C. 20315
- HERRMANN, Prof. George, Department of Applied Mechanics, School of Engineering, Stanford University, Stanford, California 94305
- HEYMAN, Dr. Jacques, Engineering Laboratories, Cambridge University, Trumpington Street, Cambridge, England
- HIGGINS, Dr. T. R., American Institute of Steel Construction, 101 Park Avenue, New York, New York 10017
- HODGKINS, E. W., Executive Secretary, Association of American Railroads, 59 East Van Buren Street, Chicago, Illinois 60605
- HOFF, Dr. Nicholas J., Head, Department of Aeronautical Engineering, Stanford University, Stanford, California 94305
- HOGGETT, Peter, Librarian, Ove Arup & Partners, Consulting Engineers, 13 Fritzroy Street, London W1, England
- HOLLENBACH, T., Director, Technical Programs, American Institute of Architects, 1735 New York Avenue, N.W., Washington, D.C. 20006
- HOLLISTER, Solomon C., Consulting Engineer, 201 Hollister Hall, Ithaca, New York 14850
- HOLT, Prof. E. C., Department of Civil Engineering, Rice University, Houston, Texas 77001
- HOLT, Marshall, 536 Charles, New Kensington, Pennsylvania 15068
- HOOPER, Ira, Seelye-Stevenson-Value-Knecht, 99 Park Avenue, New York, New York 10016
- HOOLEY, Dr. R. F., Faculty of Applied Science, University of British Columbia, Point Grey, Vancouver, B.C., Canada

- HUANG, Dr. Tseng, Engineering Mechanics Division, Arlington State College,
Arlington, Texas 76010
- HULSBOS, Dr. C. L., Chairman, Department of Civil Engineering, University of
New Mexico, Albuquerque, New Mexico 87106
- IFFLAND, Jerome S. B., Praeger-Kavanagh-Waterbury, 200 Park Avenue, New
York, New York 10017
- IRWIN, L. K., MET B-362, National Bureau of Standards, B214-PHY, Washington
D.C. 20234
- JACOBS, Gerald V., Assoc., Simpson, Stratta & Assoc., 325 5th Street, San
Francisco, California 94107
- JOHNSON, Albert L., American Iron & Steel Institute, 150 East 42nd Street,
New York, New York 10017
- JOHNSON, Eric F., Executive Secretary, American Water Works Association,
2nd Park Avenue, New York, New York 10016
- JOHNSON, A., Nora Strand 26, Danderyd, Sweden
- JOHNSON, James D., Technical Director, Steel Joist Institute, 2001 Jefferson
Davis Highway, Arlington, Virginia 22202
- JOHNSON, R. G., Canadian Institute of Steel Construction, 1815 Yonge Street,
Toronto 7, Ontario, Canada
- JOHNSTON, A. E., Executive Secretary, American Association of State Highway
Officials, 917 National Press Building, Washington, D.C. 20004
- JOHNSTON, Prof. Bruce G., 5025 E. Calle Barrill, Tucson, Arizona 85718
- JOHNSTON, L. P., Shell Oil Company, P.O. Box 193, New Orleans, Louisiana
- JUHASZ, Stephen, Newsletter Editor, Southwest Research Institute, P.O. Box
28510, San Antonio, Texas 78284
- KALNINS, Prof. A., Department of Mechanics & Mechanical Engineering, Lehigh
University, Bethlehem, Pennsylvania 18015
- KAVANAGH, Thomas C., Partner, Praeger-Kavanagh-Waterbury, Engineers-Architects
200 Park Avenue, New York, New York 10017
- KELLER, W. M., Vice-President of Research, Association of American Railroads,
3140 South Federal Street, Chicago, Illinois 60616
- KETTER, Prof. Robert L., President, State University of New York at Buffalo,
Buffalo, New York 14214
- KHAN, Fazlur R., Assoc. Partner, Skidmore, Owings & Merrill, 20 West Monroe
Street, Chicago, Illinois 60603
- KIRKLAND, W. G., American Iron & Steel Institute, 1000 16th Street, N.W.,
Washington, D.C. 20036

- KIRSTEIN, A. F., Structural Research Engineer, National Bureau of Standards,
Washington, D.C. 20234
- KLINE, Roger G., Applied Research Laboratory, MS-#92, U. S. Steel Corp.,
Monroeville, Pennsylvania 15146
- KOO, Dr. N. S., Structural Engineering Division, National Tangshan Railroad
College, Tangshan, Hopei, People's Republic of China
- KOOPMAN, K. H., Director, Welding Research Council, 345 East 47th Street,
New York, New York 10017
- KRAJCINOVIC, D., Argonne National Laboratories, Argonne, Illinois
- KRENTZ, H. A., Canadian Institute of Steel Construction, 1815 Yonge Street,
Toronto 7, Ontario, Canada
- KUENTZ, A. C., International Nickel Company, One New York Plaza, New York,
New York 10004
- LANDDECK, N. E., 3432 Sunnyside Avenue, El Paso, Texas 79904
- LANGE, Kenneth W., Manager Contract Services, Chicago Bridge & Iron Company,
901 West 22nd Street, Oak Brook, Illinois 60521
- LARSON, C. F., Assistant Director, Welding Research Council, 345 East
47th Street, New York, New York 10017
- LAURIE, J. A. P., Head, Structural Engr. Div., National Building Research
Institute, S. A. Council for Scientific & Industrial Research, P.O.
Box 395, Pretoria, South Africa
- LAY, Maxwell, Principal Research Officer, B.H.P. Melbourne Research Labs.,
P.O. Box 274, Clayton, Victoria, Australia 3168
- LECLAIR, Dean Manuel Amaya, Universidad Nacional, Facultad de Ciencias
Físicas y Matemáticas, Escuela de Ingeniería Civil, Managua,
Nicaragua, C.A.
- LEE, Prof. G. C., Department of Civil Engineering, State University of
New York at Buffalo, Buffalo, New York, 14214
- LEHMER, Gerald D., Structural Engineer, Johnson & Nielson, 7452 N. Figueroa,
Los Angeles, California
- LEMESSURIER, W. J., President, LeMessurier Associates, Inc., 711 Boylston
Street, Boston, Massachusetts 02116
- LEW, Hai S., Structures Section, Building Research Division, IAT, National
Bureau of Standards, Washington, D.C. 20234
- LIBOVE, Charles, Assoc. Prof., Mechanical Engineering, Syracuse University,
Syracuse, New York 13210
- LIM, L. C., LeMessurier Associates, Inc., 1033 Massachusetts Avenue,
Cambridge, Massachusetts 02138

- LIN, Dr. F. J., C. F. Murphy Assoc., 224 South Michigan Avenue, Chicago, Illinois 60604
- LIND, Prof. Niels C., Department of Civil Engineering, University of Waterloo, Ontario, Canada
- LLOYD, J. R., Sr. Res. Spec., Esso Production Research, Box 2189, Houston, Texas 77035
- LORIMER, A. G., Chief Architect, Port of New York Authority, 111 8th Avenue, New York, New York 10011
- LOTT, Asst. Prof. James F., Royal Military College of Canada, Kingston, Ontario, Canada
- LU, Prof. Le-Wu, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania 18015
- MAINS, Prof. R. M., Civil & Environmental Engineering Department, Washington University, St. Louis, Missouri 63130
- MALE, Milton, Resident Engineer, American Iron & Steel Institute, Oliver Building, Pittsburgh, Pennsylvania 15222
- MALO, Guillermo de Saint, Decano, Facultad de Ingenieria, Universidad de Panama, Panama City, Republic of Panama
- MANDEL, Prof. J., Civil Engineering Department, Syracuse University, Syracuse, New York 13210
- MANSOUR, A., Bridge Engineer, State Road Commission, Department of Highways, State Official Building, Salt Lake City, Utah 88411
- MARA, Paul V., Technical Director, The Aluminum Association, 750 Third Avenue, New York, New York 10017
- MAREK, Paul, Praha 6., Kozlovská la 2038, Czechoslovakia
- MARKS, Norman G., Partner, Richardson Gordon & Assocs., 3 Gateway Center, Pittsburgh, Pennsylvania 15222
- MARSHALL, P. W., Shell Oil Company, P.O. Box 127, Metairie, Louisiana 70004
- MASSONNET, C., Institut de Génie Civil, 6 Quai Banning, Université de Liège, Liège, Belgium
- MASTER, Frank M., Consulting Engineer, Modjeski & Master, 900 6th Street, P.O. Box 2345, Harrisburg, Pennsylvania 17105
- MASUR, Prof. E. F., Head, Department of Materials Engineering, University of Illinois at Chicago Circle, P.O. Box 4348, Chicago, Illinois 60680
- MCCROSKY, T. T., Secretary, American Institute of Consulting Engineers, 345 East 47th Street, New York, New York 10017

- McDONOUGH, Dr. George F., Jr., Deputy Chief, Dynamics & Flight Mechanics
Division, Aero-Astrodynamics Laboratory, George C. Marshall Space
Flight Center, Huntsville, Alabama 35812
- McDONOUGH, Prof. J., Department of Civil Engineering, University of
Cincinnati, Cincinnati, Ohio 45221
- McFALLS, R. K., Bell Telephone Laboratories, Room 4-204, North Road, Chester,
New Jersey 07930
- McIVOR, Prof. Ivor K., Department of Engineering Mechanics, University of
Michigan, Ann Arbor, Michigan 48104
- McNAMEE, Dr. B. M., Drexel Institute of Technology, 32nd & Chestnut Streets,
Philadelphia, Pennsylvania 19104
- MEITH, R. M., Chevron Oil Company, 1111 Tulane Avenue, New Orleans, Louisiana
- MICHALOS, Prof. J., Department of Civil Engineering, New York University,
University Heights, Bronx, New York 10453
- MILEK, W. A., Jr., Director of Engineering & Research, American Institute
of Steel Construction, 101 Park Avenue, New York, New York 10017
- MILLER, C. D., Chicago Bridge & Iron Company, 901 West 22nd Street, Oak
Brook, Illinois 60521
- MILLER, D. C., David C. Miller & Associates, 908 Fox Plaza, San Francisco,
California 94102
- MONTGOMERY, N. C., Butler Manufacturing Company, 6400 East 13th Street,
Kansas City, Missouri 64126
- MUKUOPADHYAY, S., Assistant Secretary (Technical), The Institution of
Engineers (India), 8 Gokhale Road, Calcutta 20, India
- MURRAY, J., Assoc. Prof., University of Oklahoma, Norman, Oklahoma
- NAPPER, Lowell A., Bethlehem Steel Company, 1601 Atlas Road, Richmond,
California 94920
- NASSETTA, A. F., Weiskopf & Pickworth, 230 Park Avenue, New York, New
York 10016
- NEWMARK, Dr. N. M., Department of Civil Engineering, University of Illinois,
Urbana, Illinois 61801
- NYLANDER, Prof. Henrik, The Royal Institute of Technology, Stockholm, Sweden
- OJALVO, Prof. Morris, Department of Civil Engineering, Ohio State University,
Columbus, Ohio 43210
- OSTAPENKO, Prof. Alexis, Fritz Engineering Laboratory, Lehigh University,
Bethlehem, Pennsylvania 18015

- PALMER, F. J., American Institute of Steel Construction, 101 Park Avenue,
New York, New York 10017
- PARMER, John F., 173 West Madison Street, Chicago, Illinois 60602
- PAULET, E. G., Room 3113 Nassif Building, Federal Highway Administration,
U. S. Dept. of Transportation, Washington, D.C. 20591
- PEKOZ, Toman, Department of Structural Engineering, Cornell University,
Ithaca, New York 14850
- PETERSEN, F. A., Metal Building Manufacturers Association, Keith Building,
Cleveland, Ohio 44115
- PETERSON, James P., Aerospace Engineer, Langley Research Center, Langley
Station, Hampton, Virginia 23365
- PFLÜGER, Prof. Dr. Ing. A., Lehrstuhl and Institut für Statik, Technische
Hochschule Hannover, Hannover-Welfengarten 1, Germany
- PFRANG, Dr. Edward O., Manager, Housing of Technology, Building Research
Division, IAT, National Bureau of Standards, Washington, D.C. 20234
- PILLAI, Dr. S. U., Reader in Civil Engineering, College of Engineering,
Trivandrum - 16, Kerala, India
- PLOFKER, Leo, Office of James Ruderman, 515 Madison Avenue, New York,
New York 10017
- POPOV, E. P., Department of Civil Engineering, University of California,
Berkeley, California 94720
- POWELL, Asst. Prof. Graham H., University of California, Dev. Sesm.,
Berkeley, California 94720
- RIMMER, Norman W., Butler Manufacturing Company, BMA Tower - Penn Valley
Park, P.O. Box 917, Kansas City, Missouri 64141
- RINGO, Prof. Boyd C., Department of Civil Engineering, University of
Cincinnati, Cincinnati, Ohio 45221
- ROBINSON, Prof. A. R., 2129C Civil Engineering Building, University of
Illinois, Urbana, Illinois 61801
- ROBINSON, Joe S., Chief Engineer, Delaware State Highway Department, Dover,
Delaware 19901
- ROBERTSON, L. E., Skilling, Helle, Christianses & Robertson, 230 Park Avenue
New York, New York 10017
- ROSSI, B. E., Society for Experimental Stress Analysis, 21 Bridge Square,
Westport, Connecticut 06880
- RUPLEY, George, McKaign, Rupley & Bahler Company, Lafayette Building,
Buffalo, New York 14203

- RUTIGLIANO, J. P., Weiskopf & Pickworth, 230 Park Avenue, New York, New York 10017
- SANDVIG, L. D., Director, Bureau of Materials, Penna. Dept. of Highways, P. O. Box 2926, Harrisburg, Pennsylvania 17120
- SCHEFFEY, Charles F., Chief, Structural Research Division, Office of Research, U. S. Dept. of Transportation, Federal Highway Administration, Washington, D.C. 20235
- SCHIER, O. B., II, Executive Director & Secretary, American Society of Mechanical Engineers, 345 East 47th Street, New York, New York 10017
- SCHILLING, Charles G., U. S. Steel Corp., Applied Research Laboratories, P.O. Box 38, Monroeville, Pennsylvania 15146
- SCHNOBRICH, Prof. William C., 3215 Civil Engineering Building, University of Illinois, Urbana, Illinois 61801
- SEARS, Frank, U. S. Dept. of Transportation, Federal Highway Administration, Washington, D.C. 20591
- SECHLER, Dr. E. E., Executive Office, Graduate Aeronautical Laboratories, California, Pasadena, California 91109
- SELBERG, Prof. Dr. Arne, Division of Steel Structures, The Norwegian Institute of Technology, 7034 Trondheim, Norway
- SFINTESCO, Duiliu, Directeur des Recherches, C.T.I.C.M., 20 Rue Jean Jaurès, 92 - Puteaux, France
- SHARP, M. L., Alcoa Research Laboratories, Box 772, New Kensington, Pennsylvania 15068
- SHAW, Prof. F. S., School of Civil Engineering, University of New South Wales, Box 1, P.O., Kensington, N.S.W., Australia
- SHERMAN, Prof. Donald R., University of Wisconsin, Milwaukee, Wisconsin 53211
- SHORE, Prof. Sidney, The Towne School of Civil & Mechanical Engineering, The University of Pennsylvania, Philadelphia, Pennsylvania 19104
- SIEV, Dr. A., Severud-Perrone-Sturm-Bandel Consulting Engineers, 415 Lexington Avenue, New York, New York 10017
- SILANO, L. G., Parsons, Brinckerhoff, Quade & Douglas, 111 John Street, New York, New York 10036
- SOLIS, Ing. Roberto, Facultad de Ingenieria, Ciudad Universitaria, Zona 12, Guatemala City, Guatemala, Centroamerica
- SIMONIS, Jack C., Babcock & Wilcox, 1562 Beeson Street, Alliance, Ohio
- SPONSLER, G. C., Executive Secretary, Division of Engineering & Industrial Research, National Research Council, 2101 Constitution Avenue, Washington, D.C. 20418

- SPRINGFIELD, John, Carruthers & Wallace Ltd., 1320 Yonge Street, Toronto,
290 Ontario, Canada
- STEIN, Dr. Manuel, M/S 245, NASA, Langley Research Center, Hampton,
Virginia 23365
- STEPHENSON, Prof. Henson K., Department of Civil Engineering, University of
Alabama, P.O. Box Z, University, Alabama 35486
- STRATING, T., Stevin Laboratorium, Afdeling Weg-enwaterbouwkunde, Delft,
The Netherlands
- SYBERT, Jack H., Chevron Oil Company, 1111 Tulane Avenue, New Orleans,
Louisiana 70112
- TALL, Prof. Lambert, Liaison Scientist, Office of Naval Research, Branch
Office, Box 39, FPO New York 09510
- THATCHER, William M., Asst. to Vice-President-Engineer, American Bridge Div.,
U. S. Steel Corp., Room 1539 N, 600 Grant St. Pittsburgh, Penna. 15230
- THOMAIDES, S. S., Engineering Department, Bethlehem Steel Corporation,
Bethlehem Pennsylvania 18016
- THOMPSON, Floyd L., Director, Langley Research Center, National Aeronautics
and Space Administration, Hampton, Virginia 23365
- THOMSEN, Prof. Kjeld, Boserupvej 216, 3050 Hamlebaek, Denmark
- THÜRLIMANN, Prof. Bruno, Pfannenstielstr. 56, 8132 Egg (ZH), Switzerland
- TIMBY, Elmer K., Partner, Howard, Needles, Tammen & Bergendoff, 1345
Avenue of the Americas, New York, New York 10018
- TOPRAC, Prof. A. A., Department of Civil Engineering, University of Texas,
Austin, Texas 78712
- TRAHAIR, N. S., University of Sydney, Sydney, Australia
- TUNG, Hsi H., Assoc. Prof., The Cooper Union, Cooper Square, New York,
New York
- TURNER, F. C., Federal Highway Administrator, Federal Highway Administration,
Washington, D.C. 20591
- TYLER, Clarence M., Jr., Metals Research Laboratories, Olin Mathieson
Chemical Corp., 91 Shelton Avenue, New Haven, Connecticut 06504
- UBBEN, J. E., American Petroleum Institute, Division of Production, 300
Corrigan Tower Building, Dallas, Texas 75201
- VANAUTRYVE, Dr. Guy, C.N.D.S.T., Royal Library, Bd. de l'Empereur 4 b - 1000,
Bruxelles, Belgium
- VERNER, Edwin, Civil & Structural Engineering, 870 Market Street, San
Francisco, California 94104

- VIEST, Dr. Ivan M., Assistant Manager, Sales Engineering Division, Bethlehem Steel Company, Bethlehem, Pennsylvania 18016
- VOGEL, Udo, Technische UnierstitHt Berlin, West Berlin, Germany
- WALMSLEY, J. L., 2229 North Main Street, Bethlehem, Pennsylvania 18017
- WANG, C. K., University of Wisconsin, Department of Civil Engineering, Madison, Wisconsin 53706
- WELDON, Harry P., J. H. Pomeroy & Company, Inc., 755 Sansome Street, San Francisco, California 94111
- WEMPNER, Dr. Gerald A., Prof. of Engineering-Mechanics, University of Alabama, Huntsville, Alabama 35807
- WESTRICH, Arthur I., Hillcrest City Club Building #11, 4900 Washington Street, Apt. 302, Hollywood, Florida 33021
- WILKES, W. J., Chief, Bridge Division, Office of Engineering & Operations, U. S. Dept. of Transportation, Federal Highway Administration, Washington, D.C. 20591
- WILTSE, Don, Executive Secretary, Structural Engineers Association of So. California, 2808 Temple Street, Los Angeles, California 90026
- WINTER, Prof. George, Department of Structural Engineering, 321 Hollister Hall, Cornell University, Ithaca, New York 14850
- WISELY, W. H., Executive Director, American Society of Civil Engineers, 345 East 47th Street, New York, New York 10017
- WOODWARD, Robert B., Managing Director, Steel Joist Institute, Crystal City Plaza, 2001 Jefferson Davis Highway, Arlington, Virginia 22202
- WRIGHT, Dr. D. T., 481 University Avenue, Toronto 2, Ontario, Canada
- WRIGHT, Prof. R. N., Department of Civil Engineering, University of Illinois, Urbana, Illinois 61801
- YEN, Prof. B. T., Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania 18015
- YU, Dr. C. K., 2520 Old St. Louis Road, Jefferson City, Missouri 65101
- YU, Prof. W. W., Department of Civil Engineering, University of Missouri at Rolla, Rolla, Missouri
- YURA, Dr. Joseph A., Department of Civil Engineering, University of Texas, Austin, Texas 78712
- ZECCA, John A., Secretary, United Engineering Trustees, Inc., United Engineering Center, 345 East 47th Street, New York, New York 10017

ADDRESSES - APPENDIX

CLARK, F.J., Lt. Gen., Chief of Engineers, Dept. of the Army, Washington,
D.C. 20315

CORTRIGHT, E. M., Langley Research Center, NASA, Hampton, Virginia 23365

DOWLINGS, J. R., Codes & Reg. Center, American Institute of Architecture,
1785 Massachusetts Avenue, N.W., Washington, D.C. 20034

EDWARDS, N. W., Pittsburgh-Des Moines Steel Company, Neville Island,
Pittsburgh, Pennsylvania 15225

FAN, S.C., Engineering Department, Bethlehem Steel Corporation, Bethlehem,
Pennsylvania 18016

HANSELL, W., Sales Engineer, Bethlehem Steel Corporation, Bethlehem,
Pennsylvania 18016

HARDESTY, E. R., Rep., American Institute of Consulting Engineers,
United Engineering Center, 345 East 47th Street, N.Y., N.Y. 10017

IYENGAR, S. H., Skidmore, Owings & Merrill, 30 West Monroe Street,
Chicago, Illinois 60603

JAMESON, W. H., Consulting Engineer, 136 East Market Street, Bethlehem,
Pennsylvania 18018

JULIAN, O. C., 40 Baystate Road, Wellesley Hills 81, Massachusetts 02181

MACADAM, J. N., Research and Technology, Armco Steel Corporation,
Middletown, Ohio 45042

NASSAR, G., c/o Arab Consulting Engrs., Tahir Square, Champohon Str.,
Cairo, Egypt

ROLF, R. L., ALCOA Research Laboratories, P. O. Box 772, New Kensington,
Pennsylvania 15068

SAMPSON, A. F., Public Buildings Service, General Service Administration,
18th & F Streets, N.W. Washington, D.C. 20401

SEIGEL, L. C., United States Steel Corp., Applied Research Labs., MS-80,
Monroeville, Pennsylvania 15146

TENG, W., Pres., Teng & Assoc., Inc., 28 E. Jackson Blvd., Chicago, Ill. 60604

VAN DER WOUDE, F., University of Tasmania, Box 252C, G.P.O., Hobart, Tasmania
7001, Australia

VAN EENAM, N., 901 Arcola Avenue, Wheaton, Maryland 20902

WEMPLE, E. L., American Institute of Consulting Engineers, 345 East 47th
Street, New York, New York 10017

By-Laws*

PURPOSES

The general purposes of the Column Research Council shall be:

1. To maintain a forum where problems relating to the design and behavior of columns and other compression elements in metal structures can be presented for evaluation and pertinent structural research problems proposed for investigation.
2. To digest critically the world's literature on structural behavior of compression elements and to study the properties of metals available for their construction, and make the results widely available to the engineering profession.
3. To organize, administer, and guide cooperative research projects in the field of compression elements, and to enlist financial support for such projects.
4. To promote publication and dissemination of original research information in the field of compression elements.
5. To study the application of the results of research projects to the design of compression elements; to develop comprehensive and consistent design formulas and rules, and to promote their adoption by specification-writing bodies.

*Revised: August 21, 1947; October 1, 1948; November 1, 1949; August 15, 1951; May 20, 1955; October 1, 1960; May 7, 1962; May 21, 1965; and May 31, 1968.

Membership

The membership of the Council shall consist of the Representatives of the Participating Organizations and a variable number of Members-at-Large.

A representative is appointed by the participating organization, subject to the approval of the Executive Committee, and continues to serve until replaced by the organization which he represents. A participating organization may appoint up to three representatives. Organizations concerned with investigation and design of metal compression members and structures may be invited by the Council to become participants.

An individual who has expressed interest in the work of the Council, and who has done or is doing work germane to its interest, may be elected Member-at-Large by the Council, following nomination by the Executive Committee.

Every three years the Chairman of the Council shall check with each Member-at-Large to determine whether he wishes to continue his membership.

Corresponding members are appointed by the Executive Committee to maintain contact with organizations in other countries that are active in areas of interest to the Council.

Meetings

The Council shall hold at least one regular annual meeting each fiscal year, and such additional meetings as may be deemed necessary by the Executive Committee. A Quorum shall consist of at least twenty members.

Fiscal Year

The fiscal year shall begin on October 1.

Duties

1. To establish policies and rules.
2. To solicit funds for the work of the Council, and to maintain a general supervision of said funds, including the appropriation of grants for specific purposes.
3. To maintain and operate a central office for the administration of the work of the Council, and for the maintenance of its records.

4. To prepare an annual budget.
5. To issue annual reports
6. To organize and oversee the committees and task groups established to carry out the projects authorized by the Council.

Officers

1. The elected officers of the Council shall be a Chairman and a Vice Chairman. The Chairman shall exercise general supervision over the business affairs of the Council, subjected to the direction of the Council, shall perform all duties incident to this office, and shall be Chairman of the Executive Committee. It shall be the duty of the Chairman to preside at meetings of the Council and of the Executive Committee. The Vice Chairman shall perform all the duties of the Chairman in his absence.
2. The terms of office of the Chairman and Vice Chairman shall begin on October 1st and shall continue for 3 years. They shall be eligible for immediate re-election for only one term of one year. In the event of a vacancy in the office of Chairman or Vice Chairman, a successor shall be appointed by the Executive Committee to serve for the remainder of the unexpired term.
3. There shall be a director engaged by the Executive Committee subject to the approval of the Council, who shall be the chief executive paid officer of the Council. Additional paid officers may be appointed by the Council as may be necessary. If there is no paid Secretary, the Chairman may appoint a Secretary, who need not be a member of the Council.
4. The Director of the Council shall conduct the regular business of the Council subject to the general supervision of the Council and of the Chairman. The Director shall be expected to attend all meetings of the Council, Executive Committee, and main committees. The Director shall be ex-officio a member of the Council and the Executive Committee. The Director shall conduct the official correspondence of the Council, shall handle the financial affairs of the Council in accordance with an approved budget, and shall keep full records thereof. He shall carefully scrutinize all expenditures and exert every effort to secure economy in the business administration of the Council, and shall personally certify to the accuracy of all bills or vouchers on which money is to be paid. He shall engage such employees as may be authorized, shall be responsible for their work, and shall determine their salaries within the budget limitations, subject to the approval of the Executive Committee. The salary of the Director and other paid officers shall be fixed by the Executive Committee. The Director shall draw up and execute all contracts authorized by the Council and its Executive Committee.

Election of Officers

1. Each year, the Executive Committee shall appoint 3 members of the Council to serve as the Nominating Committee. One of the three shall be named Chairman by the Chairman of the Council. Members of the Executive Committee or of the previous year's nominating Committee shall not be eligible to serve on the Nominating Committee.
2. The Nominating Committee shall name a slate for Chairman and Vice Chairman of the Council, and members of the Executive Committee. The Committee shall submit its nomination for Chairman and Vice Chairman to the Executive Committee prior to the Annual Meeting. Nominations for members of the Executive Committee will be submitted to the Membership at the regular Annual Meeting.
3. The election of Chairman and Vice Chairman of the Council shall be by letter ballot. The ballots shall be canvassed at the regular Annual Meeting of the Council. Should no candidate for an office receive a majority of the ballots cast for such office, the annual meeting shall elect the officer by ballot from the two candidates receiving the largest number of votes in the letter ballot.

Executive Committee

1. An Executive Committee of nine members shall be elected by the Council from its membership. The term of membership shall be for three years, and three of the members shall be elected each year at the time of the regular Annual Meeting of the Council. Nominations shall be made by the Nominating Committee as described in the section "Election of Officers". In addition the Chairman, Vice Chairman, Director, and the most recent Past Chairman and Past Vice Chairman of the Council shall be ex-officio members of the Executive Committee. Members shall take office upon their election. They shall be eligible for immediate re-election. Vacancies shall be filled by appointments by the Chairman from the membership of the Council, such appointees to serve for the remainder of the unexpired term.
2. The Executive Committee shall transact the business of the Council and shall have the following specific responsibilities and duties:
 - (a) To direct financial and business management for the Council, including the preparation of a tentative annual budget.
 - (b) To review and approve proposed research projects and Contracts.
 - (c) To appoint nominating committee.
 - (d) To appoint chairmen of committees and task groups, and approve committee and task group members.
 - (e) To review reports and manuscripts.
 - (f) To advise Council on proposed research projects.

- (g) To prepare program for Council meeting.
 - (h) To correlate and give general supervision to research projects.
 - (i) To refer inquiries relating to design practice to the Committee on Recommended Practice for definition, evaluation, and suggestions for task group assignment.
3. From time to time, the Executive Committee may ask additional consultants particularly interested in definite projects to act with it in an advisory capacity.
 4. The Chairman, with the approval of the Executive Committee, shall appoint a Finance Committee to solicit the support required to carry out its projects.
 5. The meeting of the Executive Committee shall be at the call of the Chairman or at the request in writing of two members of the Executive Committee. A quorum shall consist of five members, two of whom may be the Chairman and Vice Chairman of the Council.
 6. The Executive Committee shall transact the business of the Council subject to the following limitations:

The minutes of the Committee shall be transmitted promptly to all members of the Council. If no objection is made by any member of the Council within two weeks after the minutes have been mailed, then the acts of the Executive Committee shall be considered as approved by the Council. If disapproval of any Committee action is made by three or more Council members, then the question raised shall be submitted to the Council for vote at a meeting called for that purpose, or by letter ballot.

Contracts

The Council may make contracts or agreements, within its budget. Contracts for research projects preferably should be for the fiscal year period. Contracts with the Director or other paid employees of the Council may, with the approval of the Executive Committee, be for periods exceeding one fiscal year. At the end of such one-year period, contracts may be renewed or extended by the Council for an additional period preferably not exceeding the new fiscal year.

Standing and Special Committees

1. The Standing Committees shall be a Committee on Finance and a Committee on the "Guide to Design Criteria for Metal Compression Members". There shall be such Special Committees as may be approved by the Council.

2. Standing and Special Committees and their Chairmen, shall be appointed by, and responsible to, the Executive Committee. They shall be named at a regular annual meeting of the Council, shall take office upon appointment, shall serve for three years, and shall be eligible for immediate reappointment. Vacancies shall be filled in the same manner as regular appointments except that such appointees will complete the term of office vacated.
3. The Committee on Finance shall solicit the support required to carry on the work of the Council. The Chairman and the Vice Chairman shall be appointed from among the membership of the Executive Committee.
4. The Committee on the "Guide to Design Criteria for Metal Compression Members" shall direct the preparation and publication of the various editions of the "Guide".

Research Committees and Task Groups

1. The Executive Committee may authorize one or more research committees or task groups, each for a specific subject or field. Each committee or task group shall consist of a number of members as small as feasible for the work in hand. Members need not be members of the Council.
2. Research committee chairmen or task group chairmen shall be appointed by the Executive Committee, adequately in advance of the annual meeting of the Council.
3. All research committee or task group appointments shall expire at the time of the regular annual meeting of the Council. Prior to the annual meeting, each committee chairman or task group chairman for the ensuing year shall review the personnel of his committee or task group with the idea of providing the most effective organization, and shall make recommendations thereon to the Executive Committee. Committee or task group personnel shall be approved or modified by the Executive Committee, prior to the conclusion of the annual meeting of the Council.
4. The duties of a research committee or task group shall be:
 - (a) To review proposed research projects within its field, and to render opinions as to their suitability;
 - (b) To make recommendations as to needed research in its field;
 - (c) To give active guidance to research programs within its field, in which connection research committees or task groups are empowered to change details of programs within budget limitations;
 - (d) To make recommendations as to the time when a project within its field should be temporarily discontinued, or terminated;
 - (e) At the request of the Executive Committee to prepare summary reports covering results of research projects and/or existing knowledge on specific topics.

5. Each project handled by a research committee or task group shall be of definite scope and objective.
6. Each research committee or task group shall be responsible to the Executive Committee for organizing and carrying out its definite projects, which must be approved by the Executive Committee.
7. Each research committee or task group shall meet at least once in each fiscal year before the annual meeting of the Council, to review progress made, and to plan activities for the ensuing year.
8. Each research committee chairman or task group chairman shall make a report to the Executive Committee at the time of the Annual Meeting.

Revision of By-Laws

These By-Laws may be revised at any time upon a majority vote of the entire membership on the Council, by letter ballot or at a meeting of the Council.

Rules of Procedure

I. OUTLINE OF ROUTE OF A RESEARCH PROJECT FOR CONSIDERATION OF THE COLUMN RESEARCH COUNCIL

Projects are to be considered under three classifications:

- (1) Projects originating within the Column Research Council.
- (2) Those originating outside the Column Research Council or resulting from work at some institution and pertaining to general program of study approved by Column Research Council.
- (3) Extensions of existing CRC sponsored projects.

Projects under Class (1) are to be handled as follows:

1. Project proposed.
2. Referred to Executive Committee for study and report to Council with recommendation.
3. If considered favorably by Council, the Executive Committee will take necessary action to set up the project.
4. Project Committee, new or existing, sets up project ready for proposals and refers back to Executive Committee.
5. Executive Committee sends out project for proposals.
6. Project Committee selects and recommends successful proposal to Executive Committee for action.
7. If awarded the Project Committee supervised the project.
8. Project Chairman is to obtain adequate interim reports on project from laboratory.
9. Project Chairman advises Executive Committee adequately in advance of annual meeting as to report material available for Council presentation.
10. Executive Committee formulates program for presentation of reports annual meeting.
11. Project Committee submits reports on any completed phase of the work for the Executive Committee.

12. Executive Committee determines disposition of report subject to approval of the Council before publication.

Projects under Class (2) would be handled essentially the same except that steps 4, 5, and 6 would be omitted at the discretion of the Executive Committee. The procedure for items 7 - 12 would then be unchanged from that used for Class (1) projects.

With regard to Class (3) projects, an extension of an existing project which requires no additional funds or changes in supervisory personnel shall be approved by a majority of the Executive Committee, but need not be reported to the Council for its consideration or action. If an extension requires additional funds, such extensions may be approved by the Executive Committee subject to approval by a letter ballot from the Council.

II. OUTLINE OF A PATH OF A PROJECT THROUGH THE COUNCIL (FOR RECOMMENDED PRACTICE)

1. Task Group submits its finding to the Executive Committee.
2. Executive Committee acts and forwards to Recommended Practice Committee.
3. Recommended Practice Committee acts and forwards recommendations to Executive Committee.
4. Council votes on the matter.
5. Executive Committee transmits recommendations and findings to specification writing bodies, and/or Publications Committee arranges for publication.

III. DISTRIBUTION AND PUBLICATION OF REPORTS

For the guidance of project directors and task group chairmen the following policy is recommended with regard to the distribution of technical progress reports and with respect to the publication of reports. The scope of this procedure is intended to cover those reports that result from projects supported financially by the Column Research Council.

Distribution of Technical Progress Reports

Any duplicated report prepared by an investigator carrying out a research program may be distributed to the appropriate task group and to members of the Executive Committee with the understanding that the investigator may make further limited distribution with a view of obtaining technical advice. General distribution will only be made after approval by the task group.

Publication of Reports

Published reports fall into two categories and are to be processed as indicated.

A. Reports Constituted as Recommendations of the Council

1. The report shall be submitted to the Executive Committee which after approval will circulate copies to members of the Column Research Council.
2. Subject to approval of the Column Research Council, the Publication Committee takes steps to publish Council recommendations.

B. Technical Reports Resulting from Research Programs

1. Universities or other organizations carrying out programs of research for the Column Research Council should make their own arrangements for publications or results.
2. Assuming that the investigator wishes to arrange for such publication, approval must be obtained from the appropriate task group.
3. Reprints are currently used as means of distributing reports of projects sponsored by or of interest to the Council. Investigator should order sufficient reprints for distribution by the Council. It is assumed that ear-marked project funds will be adequate for this purpose.
4. When appropriate, reprints should be distributed under a distinctive cover.
5. A statement of sponsorship should be included in all reports.